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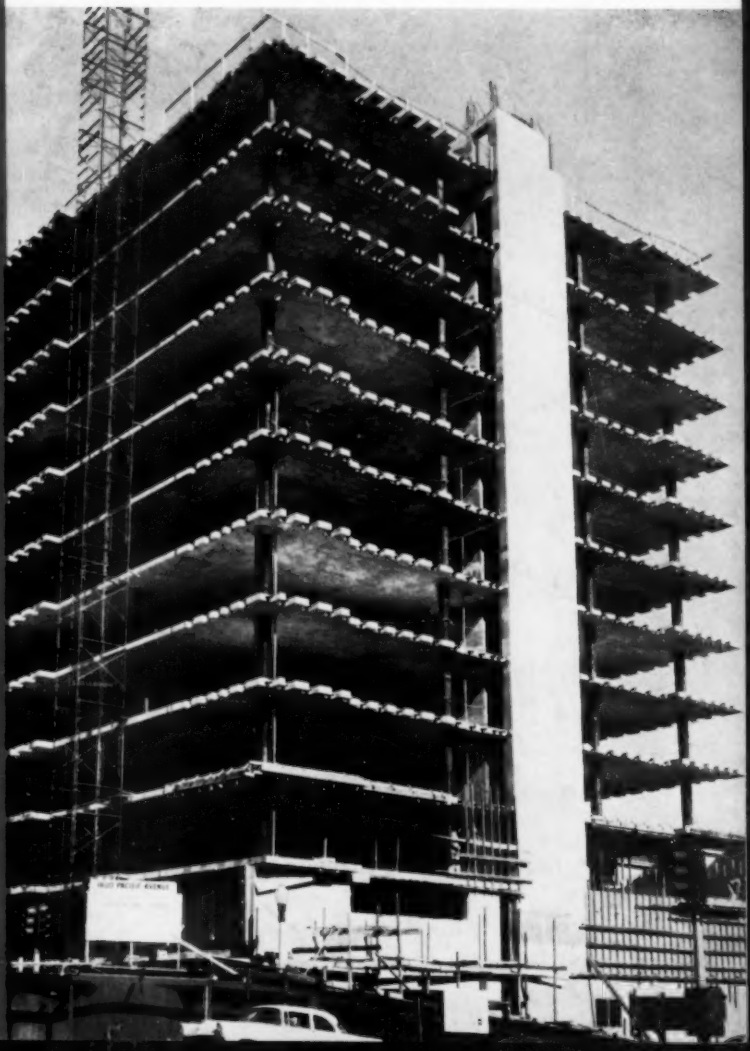

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Coming next month in the JOURNAL

PHILIP N. YOUTZ developer of the left slab construction method, describes the architectural design and construction aspects of the 12-story Huron Towers in Ann Arbor, Mich. This two-tower apartment building was designed particularly for lift slab construction.

•

"Camber in Prestressed Concrete Beams," by D. E. BRANSON and A. M. OZELL reports on an investigation to determine the initial-plus-time-dependent camber deformation in both noncomposite and composite prestressed concrete beams; necessary concrete coefficients for the analysis of total camber are presented.

•

INGE LYSE describes investigation on "Durability of Concrete in Sea Water," when exposed to freezing and thawing. Pointing out various factors of minor and major importance, the author concludes that the most important factor for the durability of concrete in sea water is the cement-water ratio combined with 10-12 percent entrained air.

•

Discussion of papers published in the October, November, and December, 1960, issues will appear in the June JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

On the Cover—A 13-story high-rise apartment building in San Francisco. The 73-unit structure features post-tensioned lift slabs with the elevator and stairway shafts slip-formed. All floors were cast on the basement slab and lifted on steel columns; they measured 105 ft x 81 ft x 8 in. and weighed about 30 tons. The stairway and elevator shafts are designed to absorb seismic shock thus eliminating need for shear walls. Owner-builder is George P. Belcher; architect: the late H. Baumann; consulting engineers: A. E. Waegemann, and Associates; prestressing consultant: T. Y. Lin.— Photo courtesy E. A. Thompson Co.

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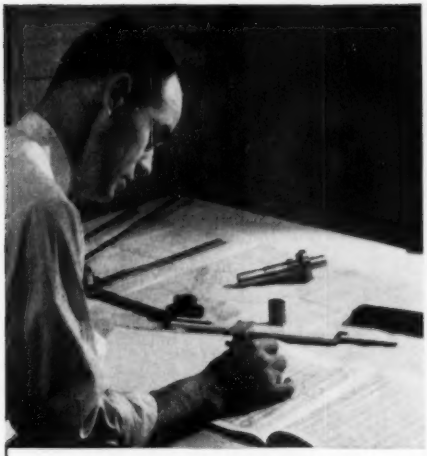

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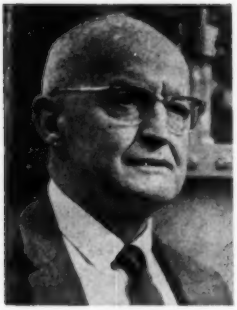

Joe W. Kelly, professor of civil engineering, University of California, Berkeley—retiring president of ACI—discusses the uniqueness of concrete pointing out that the best years of this most widely used material are still ahead, but if we are to advance successfully we will need both domestic and international cooperation.

Universal material—Cosmopolitan society

By JOE W. KELLY

■ In the American Concrete Institute, we are dealing with a unique material—a universal material—and we have a unique organization—a cosmopolitan society. I would like to discuss these in turn, and then envision how they can best go forward together.

UNIVERSAL MATERIAL

What is so unique about concrete, that 10,000 people in nearly 100 countries should join in a common effort for development and improvement of its use? There is no comparable association for other materials of construction, wherein users and individuals join with producers and service organizations as a unified membership. Nor is ACI a trade association such as those which cover many of the major aspects of concrete.

Certainly one appeal of concrete is the infinite variety of its application. It has often been called the "universal material." One would not think of using wood for a dam, or steel for a pavement, or asphalt for a building frame; but concrete is used for each of these and for many other uses of these and other construction materials. Even where another material is the principal component of a structure, concrete is usually used with it for certain portions of the work. It is used to support, to enclose, to surface, and to fill. More people need to know more about it than about the rather specialized materials.

Another basic difference between concrete and other construction materials lies in the method of manufacture. Other materials in general are made in factories (wood and stone in nature's factory) and they arrive at the point of use as finished products. Concrete arrives in the form of separate materials which are combined by the user; therefore he needs to know much about them and their use. Even for ready-mixed concrete, the user needs to prescribe the ingredients and to handle, finish, and cure the delivered concrete. Further, ready-mixed concrete plants and precasting plants are numerous and usually on a relatively small scale, so that their problems are essentially problems of the construction industry as a whole.

Still another distinctive feature of concrete is that its production requires the combined talents of many people, ranging from directors of research to ordinary workmen. Some are concerned with design, some with materials, and some with construction. Others work with properties, research, and testing; still others with equipment accessories, and special processes. All of them need to work together to produce the end result; and the consumer has an important part in shaping the properties and performance of the concrete. Concrete may be made simply by the do-it-yourself gardener, or it may be manufactured in tremendous quanities with automatic equipment and subject to precise technical control.

Characteristics

In spite of its apparent simplicity, concrete is actually a complex material. Its basic ingredient, portland cement, is of complicated non-crystalline structure which chemists are still striving to clarify. Its aggregates range from bubbles to steel punchings, through a great variety of natural and manufactured materials. Its proportions are varied in accordance with the desired quality, the demands of workability for casting, the characteristics of the available materials, and the conditions of time, temperature, and moisture which will affect the hardening. It may be fresh or hardened, plain or reinforced, mortar or mass, lightweight or heavyweight, ready-mixed or job-mixed, precast or site-cast, ornamental or utilitarian. It means many things to many people.

Concrete is not just another commodity; it has personality. Whether it is masculine or feminine I cannot say, although its occasional perversity and unpredictability suggest the latter! It will take a lot of abuse, but will sometimes talk back with evidences of maltreatment. It is responsive to constructive criticism, and unmindful of slander.

Concrete is an ideal—almost limitless in its possibilities for improvement and use; replete with faults frankly acknowledged (and herein lies its strength); and rewarding to those who treat it with proper

understanding and care. We fortunate companions who deal with it have a challenge worthy of our best efforts.

COSMOPOLITAN SOCIETY

ACI is a cosmopolitan society. It is 56 years old, having first organized in 1905 as the National Association of Cement Users, and having adopted its present name in 1913. Its organization is not unusual in most respects; it includes the conventional Board of Direction, officers, headquarters staff, committees, and local chapters. Basically it consists of 10,000 persons working voluntarily together with a common purpose, to get more done than 10,000 persons could do separately.

What then is unique about ACI? In one important respect, its membership. No longer limited to "cement users," ACI includes producers and dealers, and individuals in many diverse relations to concrete. It is not a trade association, but brings together the people and interests of many specialized fields to the common ground of concrete. Its members thresh out their common problems within the family.

Internationality

Another distinctive feature of ACI membership is its internationality. One member in four lives outside the United States. And of our members within this country, many were born abroad; in fact, we would be hard pressed indeed to replace such able and active members as any of us could name from our roster of officers and committees. The fact that so many of our colleagues are from so many fields in so many countries is a great source of gratification, not only because of the technical benefits derived but also because of social influences on general international understanding. Each has much to learn from the other, and many have much to contribute.

The program this morning is a striking example of our international interest. There are reports of official ACI representatives to recent conferences in Europe of the International Association for Bridge and Structural Engineering, the precast concrete industry, the autoclaved cellular concrete industry, and the European Concrete Committee. Director Allan Bates will describe cement and concrete plants in Russia, observed during a tour by American cement industrialists in the latest of several exchange visits by technical personnel in recent years.*

ACI has recently established a joint Collaboration Committee with the Comité Européen du Béton, or European Concrete Committee. We are members of RILEM, the International Union of Testing and Research Laboratories for Materials and Structures. We have had an official observer in the International Federation of Prestressed Concrete.

*Most of these reports were summarized in the April, 1961, News Letter—Editor.

We have been active in promoting and facilitating translations of papers and reports, both into English and from English into other languages. Many of our convention and JOURNAL papers are by overseas authors. Beginning in July of this year, papers in the ACI JOURNAL will be accompanied by synopses in French, German, and Spanish.

Recognizing the value of internationality, our Board of Direction has established a Committee on International Relations, from which fruitful recommendations are expected to further enrich the value of ideas crossing national and continental boundaries, back and forth.

Domestic cooperation

Even within the United States, ACI is committed to the philosophy that cooperation between organizations accomplishes more than narrow competition. We have six committees organized jointly with the American Society of Civil Engineers. We have official representation on several committees of the American Society for Testing Materials, and there is a clear and cordial written understanding as to the respective fields of the two organizations. We also have an official representative on one or more committees of several other organizations.

Committees

In keeping with the traditional form of democratic organization which has proved most effective, the real work of ACI is done largely by committees. These are voluntary groups of members having special competence and interest in a given aspect of concrete which we define as a "task," and who devote invaluable time and thought to its advancement. Some 50 technical committees, with about 600 individuals as members, are actively engaged in gathering, correlating, and disseminating information in accordance with our charter; and their efforts are ably supported by our Technical Director. Some committee reports have gone through ten or more drafts before they are considered to be ready for publication.

The first day-and-a-half of this convention has consisted of committee meetings, open to you all. The Technical Progress Session yesterday afternoon was an impressive sample or cross section of committee activities, problems, and plans; and the Research Committee's session scheduled for tomorrow afternoon is a forward look emphasizing the development of new concepts and techniques.

Without committees, progress would be slow indeed. We are grateful to them, individually, and to the many public-spirited organizations who make the services of their employees available. We recognize that being on an ACI committee is not just an empty honor but is an exciting challenge and an opportunity to work. The efforts of the individual committee members are unsung, but their internal satisfactions are great.

Directing all this committee activity is our Technical Activities Committee, which supervises the formation of new committees, the membership and activities of those in being, and the discontinuance of those whose task is completed. TAC also is in direct charge of our extensive program of publications and of the national meetings. It is perhaps the most distinctive feature of our organization, and it certainly has the most important influence on our policies and activities.

Chapters

Only recently did ACI become aware that its membership had grown to a point that local chapters could well be formed in many areas. Sparked by the pioneering example of our members in southern California in 1958, we now have four active chapters, a fifth approved (in Hawaii), and a number of others in process of formation. Not only can chapters have programs and projects of particular interest in their localities but also they get the benefits and pleasures of personal contacts and shop talk. And their meetings are attended by a far greater proportion of the members than that which is able to attend the national meetings.

In recognition of the growing importance of chapters, our Board of Direction has established a Committee on Chapter Activities, and our newly invested Assistant Secretary has chapter matters as one of his major responsibilities.

Staff

Behind the scenes in our united operation is the highly capable headquarters staff at Detroit. In my visits there I have been more and more impressed by its efficient organization and real interest under the leadership of Secretary Maples. Presidents come and go, but they are fortunate in having available the knowledge, counsel, and work of the continuing staff for whom our Institute affairs are their major concern.

Results

What are the products of ACI's many activities? We promote research, development, fact-finding, and writing which lead to recommended practices, codes, standards, and various other informative and useful publications. We give stimulus and support to people with ideas, and we give the weight of our influence to desirable practices in design and construction. We support better and more efficient construction. Our proceedings extending over a period of 56 years are undoubtedly the most fruitful source of information on concrete in existence, and they have been made more readily available by our new 55-Year Index. The authors of the papers in these proceedings have made invaluable contributions to progress, and they continue to lead the field with new ideas and accounts of new developments. Our meetings bring together

members and groups with common interests, and provide opportunity for discussion and planning. Our awards recognize outstanding contributions and have a stimulating effect on the membership as a whole; this year we are adding an award in memory of our late past-president, Charles S. Whitney. Our members have the satisfaction of participation and support of major improvements in the construction industry.

PROGRESS AND PROMISE

What progress has concrete made, and what is its promise? A maxim of the business world is that change is inevitable. One must either go forward or go back, at least relatively. And the field of concrete construction is no exception. Concrete must and will go forward, in quantity, quality, and economy.

The increasing volume of concrete to be cast in the future staggers the imagination. Already, in the United States alone well over 200 million cu yd are made each year, in spite of the fact that concrete has a nonrecurring market; that is, unlike bread for example, each unit of concrete used makes unnecessary another unit at the same location. The continued growth in population and the rapid per capita rise in construction of all kinds make it inevitable that concrete will be used in ever-increasing amounts. For such a gigantic task, no detail of research, design, or construction is too small to be considered and perfected.

We need only look back a few decades to see that great progress has been made, and to sense that still greater progress is to come. In the early twenties when I first became concrete-conscious, concrete was soupy and streaky, and 2000-psi concrete was respectable. Water-cement ratio was just a theory. There were no special portland cements for special purposes, no manufactured lightweight aggregates, no weight batchers, no air entrainment, and no admixtures in good standing. There was no vibration, no ready-mixed concrete, no prestressing, and little or no structural precasting. Design techniques seldom minimized or even considered the effects of temperature and moisture changes and of creep during the service life of the structure. Yet many persons closely concerned with concrete felt that the ultimate was near.

As we look ahead, what are the needs and the possibilities? I shall name a few, and you can add to the list as you wish. Some will come about through research, some through better communication, and some through greater appreciation by engineers, architects, and contractors of the importance of workmanship and inspection.

Needs and possibilities

Most of all, we need a concrete that will not crack. From my participation in many meetings devoted to practical problems of concrete, I know that the question asked most often and discussed longest is about

cracking under the complex and differing conditions to which fresh and hardened concrete are exposed. Like the common cold, cracking is always with us, and no one as yet knows how to stop it. At present, designers assume that concrete will crack, and they make provision for structural stability in spite of it. Wouldn't it be wonderful if they could rely on an uncracked section at working loads? Some day they will; already in some prestressed concrete they do.

We need a more uniform concrete. Uniformity of materials, mixing and placing, and curing conditions has been greatly improved, but much more is possible. In the ACI JOURNAL of May 1951, Boyd Mercer listed 60 variables affecting the strength of compression specimens, and many of these apply also to concrete in the structure. With greater refinement in production of materials, with increased mechanization, and with more care in workmanship, we can produce far more uniform concretes than we now do.

More economical concrete is another possibility, at least in the relative dollars of a growing economy. Inefficient practices can be eliminated and new schemes developed for lowering cost without impairing profit. Look at tilt-up and lift-slab, for example.

For some purposes, greater use of special concretes such as lightweight, prepacked, and others still to be developed is desirable. Prestressing has not nearly reached the ultimate in unified building construction. Concretes more resistant to weather, chemicals, fire, and impact are needed. Greater and faster utilization of the potential strength of portland cement will eventually come.

We need better communication, in the way of quicker translation of the findings of research into the doings of construction. Not only must the busy executive be convinced of the value of new developments, but also the job foreman and workman must be made to understand the consequences of undesirable practices. We must get the word to the worker.

On the job, we need better enforcement of principles and methods we already know, particularly on building construction and small jobs. Why, after all these years, are foremen allowed to play a hose stream into the mixer? Why let concrete flow from one corner of a house foundation to the others? Why permit segregation during placing and vibrating? Why allow concrete to dry out prematurely? Why fail to check reinforcement before it is concreted in? Why allow forms to have surface defects, or to bulge, or to fail? All of these malpractices and other simple job operations affect the quality. Competent inspection should be provided and supported, even on small jobs. There are two ways to make concrete—the right way and the wrong way. The right way costs little or no more, and it brings better results.

Tomorrow

Don't let me discourage you; we have come a long way. But we can't sit down to wait for the tortoise. The structures of tomorrow will be built largely of concrete, and it needs to be better concrete. Some 20-year men will need to be converted—men who have made concrete for 20 years and think they know all there is to know about it. Let's look 20 years ahead, not 20 years back!

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Title No. 57-62

Fire Resistance of Prestressed Concrete Beams

By L. A. ASHTON and S. C. C. BATE

Presents the results of an investigation of the fire resistance of prestressed concrete beams with post-tensioned cables, which was carried out by the Joint Fire Research Organization, Boreham Wood Hertfordshire, England, with the cooperation of the Building Research Station.

The main object of the investigation was to obtain information on the design of beams for buildings of high fire risk, such as large warehouses, where a fire resistance of 4 hr would be required for structural elements.

It is shown that, for normal forms of composite prestressed concrete given to the prestressing steel. For hard-drawn steel wire, a cover to the beams used in floor construction, resistance is governed by the protection cable of 1½ in. of concrete is sufficient to give a fire resistance of 1 hr; for longer periods of fire resistance, secondary reinforcement is necessary to hold the concrete cover in place, and a cover of 4 in. gives a fire resistance of 4 hr. By further increasing the concrete cover, fire resistance of up to 6 hr may be obtained.

The results of subsidiary tests to determine the effects of load, end-restraint, additional insulation and form of section are presented and discussed, and the effects of partial damage in fire are also considered.

■ IN COMMON WITH OTHER FORMS of construction used in buildings, prestressed concrete may be exposed to fire, the effects of which may be decisively influenced by the ability of the structural elements to limit the spread of fire and so reduce the damage to the building and its contents. Local authority bylaws in Great Britain require the elements of construction of most buildings to have a resistance to fire for a prescribed period appropriate to the size and type of building in which they are incorporated. The fire resistance of these elements is determined by test in accordance with the provisions of British Standard 476: Part 1, 1953,[1]* equipment for carrying out such tests being available at the Fire Research Station.

The first fire tests on prestressed concrete in Britain were made on small floor units,[2] and the behavior of certain types of these units, which

* The test procedure of BS 476: Part 1 does not differ significantly from that specified in ASTM E 119-58.

suffered shattering by spalling of the concrete, was the cause of concern and speculation on the possibly disastrous effects of fire on prestressed concrete structures generally. Intensified research on the resistance of prestressed concrete to fire was therefore urged by the Ministry of Works Advisory Council on Building Research to remove doubts which were hampering development. This work was undertaken jointly by the Building Research Station and the Joint Fire Research Organization of the Department of Scientific and Industrial Research.

At that time, the scanty information on the behavior of prestressed concrete in fire was almost entirely confined to that on precast members with pretensioned steel used in various proprietary forms of floor construction. It was therefore imperative that investigation should be made of the behavior of prestressed concrete beams with post-tensioned cables designed to carry heavy floor loads and to meet the requirements of local authority bylaws for periods of fire resistance up to 4 hr. The grading of 4-hr fire resistance applies in particular to large storage buildings. The investigation, which is the subject of this paper, appears to have been the first made in a systematic manner on prestressed concrete under controlled conditions. Since these tests were initiated, programs of tests on the fire resistance of prestressed concrete have been undertaken in Holland,[3] Japan,[4] and Germany.[5]

SCOPE OF THE INVESTIGATION

The program was planned to give as much information as possible on the performance in fire of prestressed concrete beams by carrying out suitable series of standard tests, leaving the basic research to be conducted as time and resources permitted. Some progress has been made in the research into the more fundamental elements of the subject,[6,7] but the present paper is confined to presenting the main results of the applied research.

The investigation was divided into three phases, and the reason for this division can best be understood if some details are given at the outset of the methods of determining fire resistance and the equipment used. The fire resistance of a structural element is measured by exposing a full-sized specimen, or a representative portion of a full-sized specimen, to defined heating conditions, and at the same time applying loads and restraints simulating those experienced in service. For beams, British Standard 476 gives a minimum requirement that the specimen shall be 10 ft long, and the equipment originally installed at the Fire Research Station was designed to test specimens of this size. Since prestressed concrete beams with post-tensioned cables of 10-ft span would not represent normal construction, and since it would be difficult to develop the proper conditions of stress in beams of large section tested on this span, a scaling technique was adopted in Phase 1 of the program to overcome the limitations imposed by the equipment then available. This technique had the object of establishing a relation between fire resistance and scale of beam which would permit extrapolation for larger beams.

In selecting a design for the specimens, it was considered that the main prestressed concrete beams in a building would usually be of composite construction

L. A. Ashton is head of the Building Materials and Structures Section of the Fire Research Station, Boreham Wood, England. Since 1940 he has been testing and doing research on the structural uses of prestressed concrete and on the properties of its concrete floors.

S. C. C. Bate is head of the Engineering Division of the Building Research Station, Watford, England. For the past 12 years he has mainly been concerned with research on the structural uses of prestressed concrete and on the properties of its constituent materials.

consisting of a prestressed concrete rib acting in conjunction with a concrete floor slab. A composite T-section was therefore chosen as the basic form for the test beams. For convenience, the design of the so-called full-sized beam was related to a clear span of 20 ft, and the test specimens in Phase 1 of the program were dimensioned by linear scaling to ½, ⅜, and ¼ full-size. From earlier research on the fire resistance of structural members in general, it appeared that the following factors might be expected to have a major influence on the behavior of prestressed concrete beams: (1) size of beam; (2) form of section beam; (3) conditions of end-restraint; (4) conditions of loading; (5) type of aggregate; (6) properties of concrete; (7) thickness of concrete cover to the prestressing steel; (8) initial stress in the prestressing steel; (9) type of steel in the tendon; (10) provision of secondary mild steel reinforcement; and (11) use of additional protection. The factors having most influence on performance were considered to be those governing the rise in temperature and the stresses in the prestressing steel and the stresses in the concrete. In all phases of the program, factors (5), (6), and (8) were kept constant and the other factors were treated as variables.

After the fire tests in Phase 1 had been completed, the opportunity arose for checking the scaling relationship obtained by testing beams nearer to full-size. A furnace larger than that at the Fire Research Station was in use at the United States National Bureau of Standards, Washington, D.C., and inquiry brought a generous offer of cooperation from the Bureau to carry out a series of fire tests to British requirements. Phase 2 of the program covered the testing of five beams of 4/5 scale and two of 1/2 scale for correlation purposes.

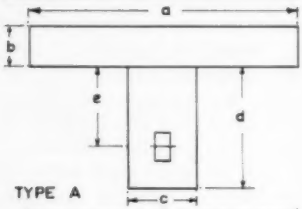
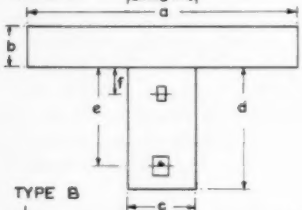
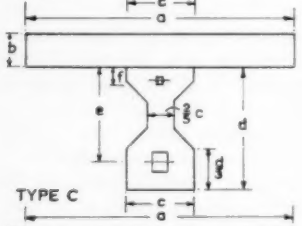
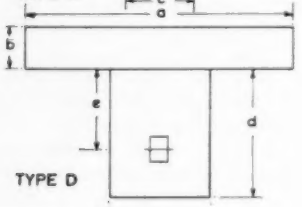
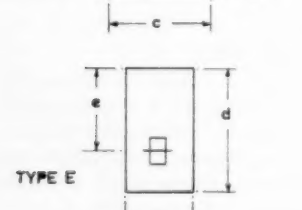
The provision of a new furnace at the Fire Research Station designed to accommodate beams having a maximum span of 24 ft made possible a further series of tests, which formed Phase 3 of the program and covered scales up to 6/5. The factors varied in Phase 3 included scale, conditions of loading, concrete cover, make of prestressing wire and provision of secondary mild steel reinforcement. The results of the tests in Phase 3 are discussed more fully here than those of the earlier phases, since they relate more directly to practice.

Scaling or model techniques have not hitherto been the subject of research for studying the fire resistance of structural elements, and this aspect of the program should only be regarded as the first step.

DESIGN AND CONSTRUCTION OF BEAMS

The design of the test specimens was based, as already stated, on a nominal full-sized beam with a clear span of 20 ft, which consisted of a prestressed concrete rib with a reinforced concrete flange cast later to represent a portion of a cast in situ floor slab in a building. For the full-sized beam, this flange was given a thickness of 6 in. to comply with the minimum requirement in local authority bylaws for floor thickness corresponding to a fire resistance of 4 hr. Following normal procedure, the beams were designed as composite sections, although the effects of composite action were ignored in allowing for losses of

TABLE 1 — DIMENSIONS OF TEST SPECIMENS

SECTION	SCALE	DIMENSIONS OF SECTIONS — in.						DETAILS OF CABLES				
								DIA. OF WIRES IN CABLES in	No. OF WIRES		CONCRETE COVER in.	
									TOP CABLE	BOTTOM CABLE	BOTTOM	SIDE
		a	b	c	d	e	f					
 TYPE A	1/4	10	1·5	2·5	4·5	3·0	-	0·10	-	8	1·0	0·9
	3/8	15	2·3	3·8	6·8	4·5	-	0·10	-	18	1·5	1·4
	1/2	20	3·0	5·0	9·0	6·0	-	0·10	-	32	2·0	1·9
	4/5	32	4·8	8·0	14·4	9·9	-	0·20	-	24	3·4	3·2
	7/5	48	7·2	12·0	21·6	14·4	-	0·20	-	56	4·4	5·2
 TYPE B	1/4	10	1·5	2·5	4·5	3·6	1·1	0·10	2	6	0·5	0·9
	1/2	20	3·0	5·0	9·0	7·3	2·3	0·10	8	24	1·0	1·9
	4/5	32	4·8	8·0	14·4	11·6	2·7	0·20	8	20	1·7	3·2
	7/5	48	7·2	12·0	21·6	17·4	5·4	0·20	8	44	1·9	5·2
 TYPE C	1/4	10	1·5	2·5	4·5	3·5	0·6	0·10	1	6	0·6	0·9
	3/8	15	2·3	3·8	6·8	5·3	0·9	0·10	2	14	0·9	1·4
	1/2	20	3·0	5·0	9·0	7·0	1·3	0·10	4	24	1·3	1·9
 TYPE D	1/2	20	3·0	7·5	9·6	6·0	-	0·10	-	32	2·6	3·2
	4/5	32	4·8	12·0	15·4	9·9	-	0·20	-	24	4·7	4·9
 TYPE E	1/2	-	-	5·0	9·0	6·0	-	0·10	-	32	2·0	1·9
	4/5	-	-	8·0	14·4	9·9	-	0·20	-	24	3·4	3·2

prestress, which were assumed to amount to 20 percent of the initial stress in the cable. Such beams are normally stressed in practice when as much as possible of the dead load of the structure is being supported, and the profiles of the cables are adjusted accordingly in design. For the test specimens and particularly for those of small size used in Phase 1 of the program, the dead load at the time of stressing was small, and consequently could not be taken into consideration in determining the positions of the cables. Thus for a single cable in a test specimen, the cover of concrete to the prestressing steel could have been greater than would occur in practice. Some of the specimens were, therefore, stressed with an additional secondary cable, which enabled the main cable to be positioned with less cover of concrete. Straight cables were used in all the beams as curved cable profiles were impracticable for the specimens to the smaller scales. All the beams were haunched near their ends, however, to increase their resistance to shear and to provide the straight cables with a cover of concrete that would conform fairly closely, throughout the span, with that for curved cables in beams of uniform section.

Beam types tested

In all, five different types of beam were tested, but, of these five types, four were variants of one main type, which was designated Type A. The dimensions and characteristics of the different types are given in Table 1. The beams of Type A were stressed with a single cable, while those of Type B, which were of similar over-all size, were stressed with two cables to provide a similar condition of prestress with a reduced cover of concrete to the prestressing steel. The effect of the shape of the section was introduced in the beams of Type C as a modification of those of Type A, the conditions of prestress being similar in each case. In the beams of Type D, the cover of concrete to the prestressing steel was increased but otherwise the form of the beams of Type A was retained. The beams of Type E consisted of the prestressed concrete rib only without the flange of the beams of Type A.

It was assumed in design that the total dead load carried by the beams incorporated in the structure would be half the live load. Thus those beams, which were required to support the maximum test load of dead load plus 1½ times live load were intended to be just free from cracking at the beginning of the fire tests. This may be seen from Table 2 which summarizes the condition of stress in the concrete under different load conditions for the different types of beams.

Throughout the investigation, the concrete was made from rapid hardening portland cement, river sand, and gravel. The aggregate, from the Thames Valley, was one that experience has shown does not behave well in fires and is representative of the type most commonly used in Britain. The composition of the aggregate is flint 89 percent, vein quartz and quartzite 2 percent. No attempt was made to scale the size of aggregate beyond a convenient reduction for the smaller beams. Thus aggregate having a maximum size of ¾ in. was used for the beams of 4/5 scale and larger, while for the ¼, ⅜, and ½ scale beams a maximum size of ⅜ in. was adopted. The specified minimum cube strengths for the beams of Phase 1 of 4500 psi at transfer and of 6000 psi at 28 days were retained for the beams in the later phases, the same quality of concrete being used for both beam and slab.

The cables were formed of 0.1 in. diameter wires in the beams of ½ scale and smaller and of 0.2-in. diameter wire in the beams of larger size. Then tensile strength was between 270,000 and 290,000 psi for the smaller size of wire and between 235,000 and 257,000 psi for the wire of larger diameter. The cables of

TABLE 2 — ESTIMATED STRESS IN CONCRETE AT TIME OF FIRE TESTS

Type of scale beam		Estimated stress in the concrete at the time of the fire tests,* psi				
		Under self-weight	Under dead load + 1½ × live load		Under dead load + live load	
		soffit of beam	Top surface of beam	Soffit of beam	Top surface of beam	Soffit of beam
A	1/4	1690	1220	−580	.....	.....
	3/8	1650	1220	−580	.....	.....
	1/2	1630	1220	−580	.....	.....
	4/5	1590	1200	−580	.....	.....
	4/5†	1380	1200	−580	900	0
	6/5	1450	1190	−600	890	−20
B	1/4	1710	1240	−590	.....	.....
	1/2	1670	1240	−590	.....	.....
	4/5†	1410	1230	−600	920	0
	6/5	1630	.....	.....	940	40
C	1/4	1680	1100	−570	.....	.....
	3/8	1730	1150	−610	.....	.....
	1/2	1630	1100	−570	.....	.....
D	1/2	840	960	−600	.....	.....
	4/5	760	980	−600	.....	.....
E	1/2	1730	2430	−580	.....	.....
	4/5	1700	2340	−580	.....	.....

* Columns are left blank where the conditions were not imposed in the fire tests
† These beams had a span of 24 ft, all other beams had spans correct for their scales.

TABLE 3 — SUMMARY OF RESULTS OF STRUCTURAL TESTS

Type of beam	Scale	Design load, kips	Dead load + 1½ × live load, kips	Load at start of cracking, kips	Load at failure, kips	Age at test, days
A	1/4	4.8	6.4	6.2	12.1	99
	3/8	10.8	14.4	15.5	27.1	95
	1/2	19.3	25.6	27.6	48.2	108
	4/5	48.2	64.3	65.9	112.7	40
	3/8*	10.8	.....	.....	22.2	.....
	1/2*	19.3	.....	.....	37.6	.....
B	1/4	4.9	6.5	7.5	12.5	335
	1/2	19.5	26.0	26.0	48.4	346
				26.7	45.9	650
C	1/4	4.4	5.8	5.6	11.0	293
	3/8	10.3	13.7	15.2	27.3	309
	1/2	17.5	23.3	25.8	43.9	291
D	1/2	18.8	25.1	24.9	42.6	43
	4/5	48.8	65.1	62.3	115.4	39
E	1/2	8.2	10.9	11.9	18.6	55
	4/5	21.3	28.4	29.8	47.3	48
				31.6	51.3	210

* Tested after heating for a period shorter than that necessary to cause failure in the fire tests (See Fig. 9)

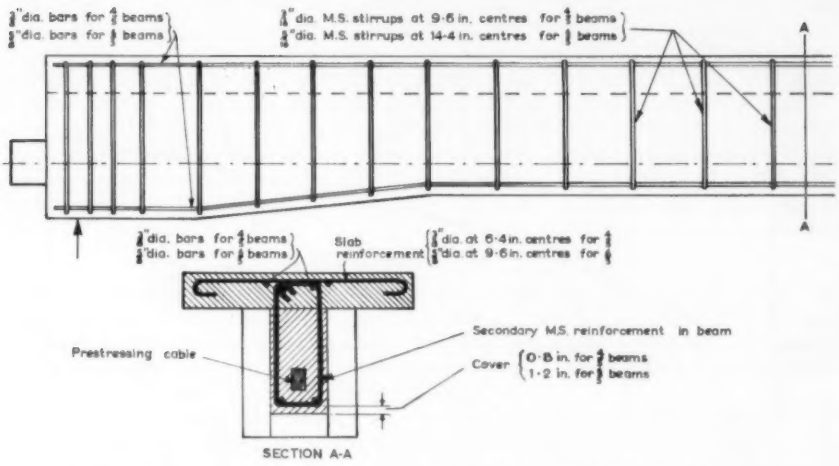

⅜"dia. bars for ⅔ beams
⅜"dia. bars for ½ beams
¼"dia. M.S. stirrups at 9·6 in. centres for ⅔ beams
¼"dia. M.S. stirrups at 14·4 in. centres for ½ beams
A
A
¼"dia. bars for ⅔ beams
⅜"dia. bars for ½ beams
Slab reinforcement
¼"dia. at 6·4 in. centres for ⅔
⅜"dia. at 9·6 in. centres for ½
Secondary M.S. reinforcement in beam
Prestressing cable
Cover 0·8 in. for ⅔ beams
1·2 in. for ½ beams
SECTION A-A

Fig. 1 — Type A beam showing secondary mild steel (M.S.) reinforcement

0.1 in. diameter wire were stressed by a method specially devised to compensate for the slip of the wires in the anchorage. Wires were stressed initially in pairs or in groups of four and anchored, and finally the whole cable was restressed and shims inserted behind one anchorage to take up the slip in the wedge grips.

The standard Magnel-Blaton method of stressing was used for the wires of 0.2 in. diameter which were temporarily overstressed to offset the slip of the wires at the anchorage. In each case the initial stress in the steel was approximately 0.6 times the tensile strength of the wire.

After the completion of stressing, the cable cavities were injected with grout using a power driven pump. The grout, which consisted of rapid hardening portland cement with a W/C ratio of 0.4, was pumped in through an access hole at one end of the beam while the other end of the beam was raised to ensure expulsion of air through an outlet hole at that end.

Some of the beams of Phases 1 and 2 were provided with additional protection to their sides and soffits of vermiculite concrete consisting of cement and exfoliated vermiculite in the proportions of 1:4 by volume. This insulating material was made up in the form of slabs varying in thickness from 1 to 2½ in., which were reinforced with chicken wire mesh and were used to line the mold before the beams were cast.

Secondary mild steel reinforcement was introduced in some of the beams tested in Phase 3 of the program. This reinforcement, which was additional to that in the anchorage region and in the flange, is shown in Fig. 1. As an alternative, one of the beams of 4/5 scale was reinforced with welded wire fabric with wires of 0.13 in. diameter at 6-in. centers in each direction. The cover of concrete to this reinforcement was 0.8 and 1.2 in. for the beams of 4/5 and 6/5 scale, respectively, which is equivalent to a cover of 1 in. for the full-sized beam.

Thermocouples were attached to selected wires of all beams, and were also located in the concrete at different depths from the surface in some beams for measuring temperature during the fire tests.

All the beams were manufactured at the Building Research Station, except

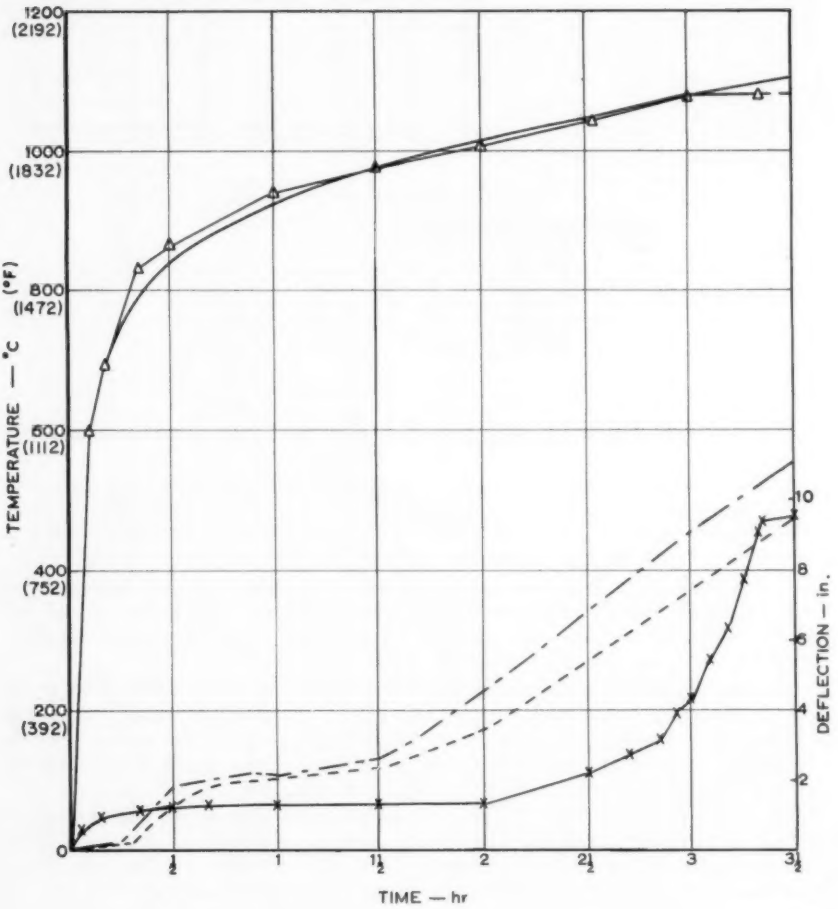


Fig. 2—Recorded temperatures and deflection for a fire test on Type A beam of 4/5 scale with secondary reinforcement

where, owing to difficulties in handling, the slabs for the beams of 6/5 scale were cast after the prestressed concrete elements had been transported to the Fire Research Station. The specimens for testing in the United States were crated and shipped as deck cargo, and suffered no damage in transit. The total number of beams made in all sizes included sufficient specimens for structural tests on each size and type which could be accommodated in the testing equipment at the Building Research Station. A summary of the results of these static loading tests, in which load was applied equally at four points, as in the fire tests, is given in Table 3.

TEST CONDITIONS

The procedure followed in the fire tests was that prescribed in British Standard 476. A standard rate of heating defined by a time-temperature curve is specified for all fire resistance tests and the furnace temperature must attain the prescribed values within certain limits at the given times. The curve is shown superimposed on test results in Fig. 2.

The sides and soffit of the beam are exposed to the furnace, the test representing the conditions of a fire on the underside of a floor. For structural elements such as beams, which have only a load-bearing function in a building, the fire resistance is determined by the time for which a specimen can resist collapse in the test. The grade of fire resistance attributed to a specimen is that period of the following which is nearest to, but lower than, the test duration: ½, 1, 2, 4, and 6 hr. In the 1932 edition of British Standard 476 which was in force when the first two phases of the program were carried out, it was specified that the test load should be 1½ times the design load. This has been interpreted as meaning that the test load shall be the dead load plus 1½ times the live load.

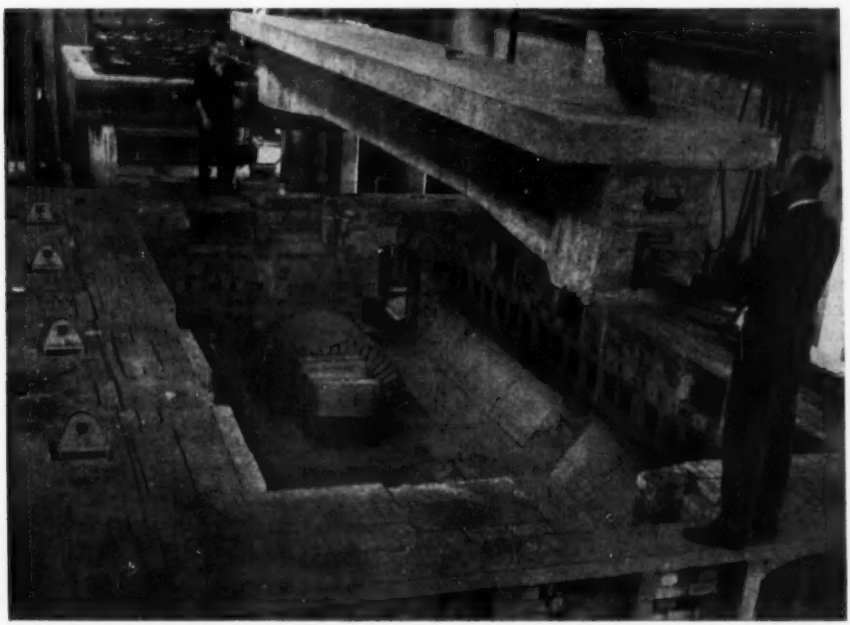


Fig. 3 — Floor furnace with 24 ft span Type B beam being placed in position

Fig. 4 — A 24 ft span 4/5 scale beam on furnace, ready for test

In the current edition (1953) of British Standard 476, the test load is required to be that load which develops in the specimen stresses of the same magnitude as those contemplated in the design. The majority of beams in Phase 3 of the program were tested with the design load to measure their fire resistance in accordance with the present Standard, but a sufficient number of beams were tested with 1½ times the design load (as interpreted above) for the purpose of completing the work on scaling.

In all phases of the program the load was applied by hydraulic jacks at four points on the beam, i.e., at ⅛, ⅜, ⅝, and ⅞ points of the span, each point taking one quarter of the total load. Two beams of Type A of each scale in Phase 1, however, were tested without any imposed load, and one ½-scale beam with just sufficient load to induce stresses in the concrete equal to those in the full-scale beams due to dead load alone. Except for two beams of Type A of each scale in Phase 1, all beams were simply supported on steel brackets fixed to the loading frame without additional end-restraint. The exceptions were restrained against longitudinal expansion during the fire test.

The fire resistance of a beam is affected to a marked extent by the amount of free water in the concrete, and it was therefore important to cure the specimens until the moisture content had been reduced to a reasonable value, representing the condition which might be expected in service. For the tests made in Britain, the unprotected beams were matured in the laboratory without special facilities for conditioning; the beams with vermiculite encasements were stored until required at a constant temperature of 64 F and at a constant relative humidity of 65 percent. The age at test of the small beams (½ scale and lower) varied from 6 months to more than a year, while the age of the large beams in Phase 3 varied between about 2 years and 2 years 8 months. The beams sent to the United States were about 8 months old when tested. After arrival at the Bureau of Standards in Washington the beams were conditioned for a minimum period of about 8 weeks in an atmosphere of approximately 55 per cent relative humidity at a temperature of 70 F.

The furnace in use when Phase 1 of the program was carried out was a relatively small one which has been described elsewhere.[8] In 1956 a new furnace was installed (Fig. 3 and 4) replacing the old one and designed to accommodate beams of 24-ft span. The furnace is, in effect a box open at the top, and the structure to be tested is placed on the walls so that its underside is exposed to heat. When a beam having a narrow flange is being tested, the opening in the

top of the furnace on either side of the flange is closed with refractory concrete covers and an asbestos rope seal, which prevents loss of heat but permits the beam to deflect freely.

Thermocouples for measuring the furnace temperature were located at points along the beam with their hot junctions 3 in. from the surface of the beam. The furnace has burners along two of its sides and the fuel supply of premixed town's gas and air is controlled to give the time-temperature relationship specified in British Standard 476 within the permitted tolerances. A continuous record of furnace temperature was obtained by recorders and the temperatures in the beams were measured by multipoint indicators. The vertical movement of the beams at midpoint in the span was measured by a simple form of deflectometer.

BEHAVIOR OF BEAMS IN THE FIRE TESTS

A typical set of results obtained in the fire tests is summarized in Fig. 2. During the first few minutes of the test on a loaded beam, there was a relatively rapid increase in deflection but otherwise little apparent damage to the concrete. With the larger beams tested in Phases 2 and 3, a quantity of water sometimes escaped from the beam into the furnace during this stage. Where later examination of the cable duct was possible, it showed that there were no pockets in the grout which could have accommodated this quantity of water; it is thought that this water originated in the surrounding hotter concrete where it evaporated, that it condensed in cracks and crevices in the region of the cable, which was then at a temperature appreciably below 212 F and that it finally escaped from cracks caused by the combined effects of heat and applied load. In some of the beams water emerged at the anchorages after about the same time interval, but in all cases, this flow of water ceased before the temperature in the cable reached 212 F. It should be noted that no sign of free water was found in ducts of similar beams subjected to structural tests alone.

After the early increase, the deflection of the beam remained almost unchanged for much of the remainder of the test, although considerable visible cracking and crazing of the concrete developed and some spalling of concrete at arrises occurred. During this stage, the temperature of the steel rose to about 212 F, where it remained for a period, during which the concrete and grout surrounding the steel dried out completely. Later, for beams failing primarily in flexure without other major damage, the temperature rose further and when the average temperature was between 392 and 572 F, the deflection of the beam again began to increase. More superficial damage to concrete by cracking and spalling occurred, and finally, when the average temperature of the steel reached about 752 F, the beam collapsed.

The tests showed that there was a tendency for the mode of failure to change with increasing size of beam. For loaded beams of ½ scale and smaller, failure was the result of fracture or yield of the steel. For

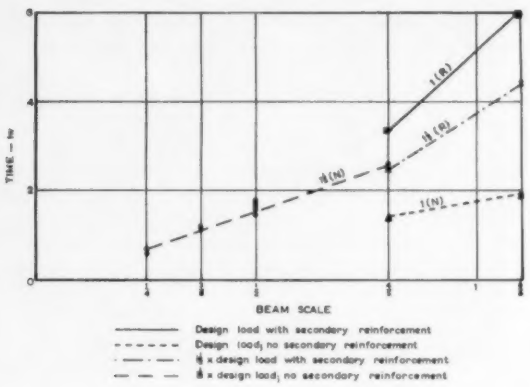

TIME — hr
BEAM SCALE
Design load with secondary reinforcement
Design load, no secondary reinforcement
½ x design load with secondary reinforcement
½ x design load, no secondary reinforcement

Fig. 5 — Scale of beam and fire resistance

beams of 4/5 scale in Phase 2, yield and fracture of the steel was again the primary cause of failure, but, just before collapse, the concrete cover to the prestressing steel became dislodged. The beams of 24-ft span in this scale tested in Phase 3 required secondary mild steel reinforcement to ensure that the cover did not collapse and lead to premature failure of the beam, again by yield of the steel. The beams of 6/5 scale required secondary mild steel reinforcement not only to prevent the early loss of the concrete cover to the cables but also to avoid shear failure along the junction between the beam and the slab. This qualitative result clearly demonstrates the limitations of scaling techniques and the advisability of determining fire resistance by tests on full-scale structures. A summary of the major quantitative results of the fire tests is presented in Tables 4, 5, and 6, and in Fig. 5.

DISCUSSION OF RESULTS

Effect of load and end-restraint

In the main series of tests, the beams were tested under load, which was initially the dead load plus 1½ times the live load, and later the dead load plus the live load to comply with the previous and present issues, respectively, of British Standard 476. The relative values of the dead and live load were arbitrarily selected in designing the specimens, and it is therefore most satisfactory to consider the loading conditions in relation to the bending tensile stresses induced in the concrete. For the more severe conditions of loading, the bending tensile stress in the concrete was between 570 and 610 psi, while for the less severe condition the stresses at the soffits of the beam varied between 40 psi compression and 20 psi tension for the different specimens. Since the start of the investigation there has been a trend towards the use of higher bending tensile stresses under working loads; the Code of Practice, CP 115: 1959[9] permits a bending tensile stress of 200 psi for loading of long duration

Fig. 6 — A ½ scale beam after test without applied load

for concrete of the quality used in the specimens and under special circumstances allows this figure to be increased to 450 psi. Thus the actual conditions of working load that may be encountered in present design lie between the two conditions of load applied in the tests.

The results in Table 4 and Fig. 5 for the beams of Type A, tested on a span of 24 ft with scales of 4/5 and 6/5 and reinforced with additional mild steel, show that the time for failure under the more severe test loading was about three quarters of that under the less severe loading. The comparable results for beams of Type B of 4/5 scale show a smaller difference of about 10 percent.

The most severe conditions of compressive stress in the concrete for beams of T-section, such as those in this investigation, occur when the beam is unloaded. Phase 1 included tests on beams of Type A with scales of ¼, ⅜, and ½, in which the behavior of loaded and unloaded beams was compared with and without end-restraint. The results of these tests are summarized in Table 5. The two smaller sizes of beam tested without applied load did not collapse and the results are therefore of little interest, but the beams of ½ scale both with and without end-restraint failed by crushing of the weakened concrete at the soffit of the beam under the influence of the prestressing force remaining in the cable; deflection at failure was upward as shown in Fig. 6. The condition of compressive stress in the concrete at the lower surface of these beams is considered to be more severe in the tests than would normally be experienced in practice. A further beam of ½ scale was tested without end-restraint under a small applied load to represent a more normal condition of stress under dead load as shown in Table 5. During the early stages of the test on this beam, it deformed upward by as much as 0.3 in. at midspan showing the predominant influence of the prestressing force on deflection at this stage, but when the test was stopped after 2 hr, the deflection had reversed and was 0.4 in. downward showing that the steel in the prestressing cable had crept and that the beam was deforming under the applied load. For unloaded or lightly loaded construction, any tendency for upward deflection to occur in building fires would usually be prevented in practice by restraint from the surrounding structure, so that the risk of reduced fire resistance on this account may be discounted. The experimental data show that the fire resistance of prestressed concrete members, as

TABLE 4 — TIMES TO FAILURE FOR PRESTRESSED CONCRETE BEAMS IN FIRE TESTS — SUMMARY OF MAIN PROGRAM

Type of beam	Secondary reinforcement	Additional protection	Load for test, DL = dead load LL = live load	Time to failure, hr-min, for beam scaled to: 1/4	3/8	1/2	4/5 (16 ft span)	4/5 (24 ft span)	6/5	Remarks
A	None	None	DL + 1½ LL	0-33 0-40	1-06 1-08	1-38 1-46	2-35 (a) (U.S.)			(a) Failure after collapse of cover
						1-36				Beam made and tested at same time as beam in U.S. tests
						1-28 (U.S.)				
	Mild steel stirrups and carrier bars	None	DL + 1½ LL	....	....	....	....	2-30	4-23	
	None	None	DL +LL	....	....	....	....	1-26 (b)	1-54 (c)	(b) Failure after collapse of cover (c) Failure by shear between beam and slab
	Mild steel stirrups and carrier bars	None	DL +LL	....	....	....	....	3-22	6-00	
	Wire mesh of 0.13 in. diameter at 6-in. centers	None	DL +LL	....	....	....	....	3-23	....	
	None	1 in. of vermiculite concrete	DL + 1½ LL	2-34	2-38 (d) 3-37	3-51	6-03 (e) (U.S.)			(d) Concrete spalled near one support after 1 hr 15 min and later shear failure developed at this point (e) Beam had not failed when test was stopped
B	None	None	DL + 1½ LL	0-35		1-20	....	....	....	
	Mild steel stirrups and carrier bars	None	DL + 1½ LL	....	....	....	....	2-41		

	None	None	DL + LL	....	....	....	....	1-02 (f)	1-37 (g)	(f) Failure after collapse of cover (g) Failure by shear between beam and slab
	Mild steel stirrups and carrier bars	None	DL + LL	....	....	....	....	3-00	4-57	
	None	1 in. of vermiculite concrete	DL + LL	3-25	....	6-00	....	....	....	Neither beam had failed when tests were stopped
C	None	None	DL + 1½ LL	0-30	0-50	1-16 (h)	....	....	....	(h) Beam failed in shear
D	None	None	DL + 1½ LL	....	....	3-00	3-40 (i) (U.S.)	....	....	(i) Failure after collapse of cover
E	None	None	DL + 1½ LL	....	....	0-55	1-55 (j) (U.S.)	....	....	(j) Failure after collapse of cover
	None	1 in. of vermiculite concrete	DL + 1½ LL	....	....	....	4-39 (U.S.)	....	....	

Notes: (U.S.) denotes tests carried out at the National Bureau of Standards, Washington, D.C. Except where noted in the table, all beams failed in flexure.

with reinforced concrete or protected structural steel members, increases with the reduction of the load below its full design value.

When a beam in a structure is exposed to fire, there may develop considerable restraint to longitudinal expansion. In reinforced concrete or in properly protected steel construction, longitudinal restraint tends to reduce deformation and increase fire resistance. The results in Table 5 show that the fire resistance of the loaded and restrained prestressed concrete beams of ¼ and ⅜ scale was higher by 58 and 26 percent, respectively, than that of the comparable beams without end-restraint. For the beams of ½ scale, the introduction of restraint reduced the fire resistance of loaded beams by 16 percent. No attempt was made to introduce restraint in the tests on beams of larger size than ½ scale, so that it is not known whether the fire resistance for this condition would continue to diminish with respect to that for the condition of simple support as the size of member increased.

Effects of concrete cover to the prestressing steel

For beams of composite construction, such as the majority of those tested in this investigation, failure under load at normal temperatures would result from fracture or excessive elongation of the steel. The ultimate strength of these beams is therefore primarily governed by the tensile strength of the steel. As the design load applied in the fire tests is about half the normal failing load for these beams, it would

Fig. 7 — Part of a 4/5 scale beam with additional mild steel reinforcement after test

be expected that failure in the fire tests would occur when the tensile strength of the steel was reduced to about half that at room temperature. It has been shown[6] that the types of hard-drawn steel wire commercially available in Britain for prestressing lose about half their strength when heated from room temperature to 752 F, and therefore failure of the prestressed concrete beams would be expected when the mean temperature of the cable reached this value. The measured temperature of the wires during the tests, such as those illustrated in Fig. 2, confirmed that collapse occurred shortly after the mean temperature in the cable reached 752 F. The problem of ensuring that a prestressed concrete beam of this type has adequate fire resistance is therefore one of providing sufficient protection, to delay, for the appropriate period, the rise in temperature of the wires to this critical value. Protection can normally be provided by the grout in the cable duct and the concrete cover surrounding the duct.

The results in Table 4 show that, although concrete is a relatively poor insulator, it can provide protection to the prestressing steel in prestressed concrete beams subjected to fire for periods up to 6 hr. Detailed examination of the results of the tests on beams Types A to D inclusive, in relation to the thickness of concrete cover provided, suggests that the following amounts of cover should be used to enable beams to comply with the fire resistance requirements for the types of building covered by local authority bylaws:

Grade, hr	Thickness of concrete cover, in.
½	1
1	1 ½
2	2 ½
4	4

It must be emphasized, however, that some compromise, when considering the effects of loading and of distribution of the prestressing steel, has been necessary in reaching this simplified conclusion. The

Fig. 8 — Part of a 4/5 scale beam without mild steel reinforcement after test

adoption of these thicknesses of cover in design should usually ensure that the fire resistance of beams forming part of a floor construction will not be less than the corresponding grade.

In the fire tests on some of the beams of smaller size, without additional reinforcement, the concrete cover remained in position for periods considerably in excess of 1 hr but, in the tests on some of the larger beams, fire resistances much in excess of 1 hr were only obtained when the concrete cover to the prestressing steel was held in position by additional mild steel reinforcement, the effect of which is shown by comparing Fig. 7 and 8. In most of the beams tested, this reinforcement consisted of a nominal amount of steel in the form of closed stirrups

TABLE 5 — TIMES TO FAILURE FOR TYPE A PRESTRESSED CONCRETE BEAMS IN FIRE TESTS WITH DIFFERENT CONDITIONS OF LOADING AND END-RESTRAINT

Condition of loading and end-restraint	Stress in the concrete at the soffit of the beam at the start of the test, psi	Time to failure, hr-min, for beams scaled to:		
		1/4	3/8	1/2
Loaded and unrestrained	—580	0-36 (a) 0-40 (a)	1-06 (a) 1-08 (a)	1-38 (a) 1-46 (b)
Loaded and restrained	—580	0-57 (a)	1-24 (a)	1-29 (a)
Unloaded and unrestrained	1660 (average for the three sizes)	1-05 (d)	1-25 (d)	1-10 (c)
Unloaded and restrained	1660 (average for the three sizes)	1-18 (d)	1-48 (d)	1-02 (c)
Partially loaded and unrestrained	1140	—	—	2-00 (e)

Notes: (a) Collapse due to yield or fracture of the steel
(b) Failure due to shear of the concrete near one end
(c) Failure with upward deflection due to failure of the concrete at the soffit of the beam
(d) Collapse of concrete cover only
(e) No failure, fire test discontinued

fixed to carrier bars at top and bottom to produce a cage sufficiently rigid for easy handling. The spacing of the stirrups was arbitrarily chosen to be approximately the same as the depth of the cable in the prestressed portion of the beam. With a concrete cover of 0.8 in. for the beams of 4/5 scale and 1.2 in. for the beams of 6/5 scale, this arrangement of reinforcement was fully effective in retaining the concrete in position. The beam of 4/5 scale with the alternative form of reinforcement consisting of a reinforcing fabric made up of 0.13 in. diameter wires also gave satisfactory results in the fire tests as shown in Table 4; the weight of steel in the fabric was less than that in the cage but the correct positioning and fixing of the cage of reinforcement was much easier.

Effect of additional insulation

The influence of protective finishes applied to the exposed surfaces of the test beams was investigated by testing beams protected by different thicknesses of vermiculite concrete. The vermiculite concrete slabs, which were reinforced with a light wire mesh, were precast and used to line the molds for the beams. In the first phase, thicknesses of 1, 2, and 2½ in. were used to protect the beams. The insulation was so effective that the thinnest encasement was found to increase fire resistance sufficiently for practical purposes. Thus a beam of Type A of ½ scale with an encasement consisting of a 1 in. thickness of vermiculite concrete had a fire resistance of 3 hr 51 min which is more than 2 hr greater than that for its unencased counterpart as shown in Table 4; similar beams with encasements of vermiculite concrete of 2 and 2½ in. endured 6 hr without failure. The fire tests on beams of Types A and E of 4/5 scale included beams protected with 1 in. of vermiculite concrete, and for each type the fire resistance was increased by more than 2½ hr.

One unexpected feature of the tests on beams with additional protection was the explosive spalling of the concrete which occurred in the tests on both the beams of 4/5 scale and in one beam of ⅜ scale. The spalling dislodged the vermiculite slabs, large pieces of which fell from the soffit of the beams in the region of the haunch. The most probable explanation of this phenomenon is that the insulating slabs prevented the concrete of the beams drying sufficiently, and the resulting moisture content produced those conditions favorable to spalling, which are familiar with unprotected concrete.

The type of construction examined in this investigation would not normally have decorative or other finishes. The application of an insulating material to improve fire resistance is, therefore, only justified if it provides appreciably better insulation than the same thickness of concrete. The vermiculite concrete slabs, which were used for most of the beams with additional protection, provided insulation in the fire tests equivalent to twice their thickness of concrete. The results of other

fire tests have shown that the insulation given by sanded-gypsum plaster is not sufficiently better than that of concrete to be of use. The value of other insulating materials as protection can be assessed relatively easily by test, and it has been found, for example, that ⅞ in. of vermiculite gypsum plaster or ¾ in. of sprayed asbestos corresponds to 1 in. of the vermiculite concrete slabs in the protection given. Both these materials have been found to have satisfactory adhesion to concrete in fire tests of up to 4 hr duration when applied to concrete surfaces cast against wood forms.

Effect of form of section

The form of the cross section of the beams was varied in three ways: the first variation was to split the prestressing cable into two in the beams of Type B to reduce the concrete cover to the prestressing steel at the soffit of the beam; the shape of the section was changed in the beams of Type C to determine whether the introduction of a relatively thin web in the section would lead to explosive spalling of the concrete; and finally the proportion of prestressing steel in the section was varied in effect in the beams of Type E by omitting the cast in situ flange.

Separation of the main cable into two parts generally reduced the fire resistance of the beams without additional protection, as shown in Table 4. The reduction was not, however, quite as great as had been expected; the information obtained from the tests on the beams of Type A had suggested that the smaller cover of concrete to the main cables in the beams of Type B would have led to collapse at a slightly earlier stage, but the secondary cable, which was relatively well protected by concrete and so little damaged by the effects of the fire test, apparently made an appreciable contribution to the resistance of the beam. Where the beams of Type B of ¼ and ½ scale had additional protection of vermiculite concrete, their fire resistance was greater than that of the comparable beams of Type A; the additional protection appeared to enhance the part played by the secondary cable.

No explosive spalling of the concrete was experienced in the tests on the beams of Type C, even though in the beams of ½ scale the thickness of the web was only 2 in. which has been found to be a critical thickness for explosive spalling in some beams with pretensioned steel. The fire resistance of these beams was primarily governed by the cover of concrete to the steel and the results of the tests were consistent with those for beams of Types A and D.

In the beams of Types A to D, inclusive, the provision of the top flange ensured that the compression zone in the concrete at failure was remote from the fire and that the loss of strength of the concrete in this region was small; as a result, the lever arm at failure in the fire tests was little influenced by the heating condition, and the reduction in the ultimate resistance of the beam was almost completely defined

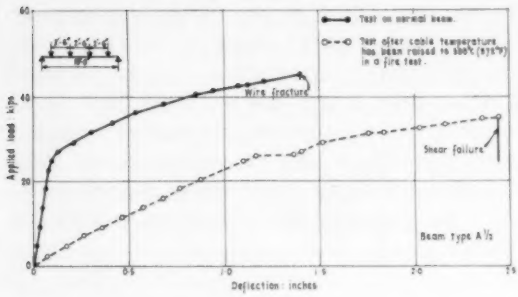


Fig. 9 — Effect of fire on behavior of prestressed concrete beam Type A, ½ scale

by the reduction in the strength of the steel during the test. In the beams of Type E, the concrete in the compression zone was severely heated and its strength appreciably reduced during the test, so that at failure the ultimate resistance moment was defined by a reduced lever arm as well as the reduction in strength of the steel. Consequently with such sections, design for fire resistance cannot be based on considerations of the protection afforded to the prestressing steel alone, as may be seen by comparing the results for beams of Type A and E in Table 4. Beams of Type E would not normally be used in buildings and the situation envisaged in these fire tests seldom arises in practice, but a more adverse situation, partly analagous to that of the beams tested without load, might arise with continuous beams of T-section. At the points of support for such beams, the concrete in the compression zone of the section might be completely exposed to fire, while the prestressing steel was relatively well protected, and, although some redistribution of moment would occur, fire resistance would probably be less than for comparable simply supported beams.

Effects of partial damage in a fire

The investigation was mainly concerned with the conditions leading to collapse in fire and with the measures that should be adopted in design to attain the different grades of fire resistance. Secondary problems of practical importance however are the assessment of damage to prestressed concrete beams which have been exposed to fires of insufficient severity to cause collapse, and possible ways of minimizing such damage. To examine these problems, a beam of Type A of ½ scale was subjected to a fire test in which the temperature of the prestressing steel only reached 572 F, so that collapse did not occur; after the beam had cooled, it was transferred to a testing machine for a structural test. The relationship between load and deflection for this beam is compared with that for an undamaged beam in Fig. 4, which shows that the prestress had completely disappeared but that although there was a shear failure the reduction in strength was not unduly severe. It was, therefore, concluded that creep of the steel was the main cause of the loss of prestress.

TABLE 6 — RESULTS OF RELOADING TESTS AT ROOM TEMPERATURES ON BEAMS OF TYPE A OF 4/5 SCALE AFTER HEATING FOR PERIODS SHORTER THAN WOULD CAUSE COLLAPSE

Wire Ref.	Final temperature of cable, F		Central deflection, in.		Behavior in reloading test		
	Maximum	Mean	At end of test	After 48 hr (without load)	Deflection under dead load + live load, in.	Deflection under dead load + 1½ × live load, in.	Failing load, kips
E	388	363	1.46	−0.15	0.33	0.70	87.4
F	374	365	1.60	−0.07	0.59	1.21	81.3
G	383	370	1.41	−0.05	0.44	0.82	82.7
H	536	513	1.22	0.00	1.03	1.66	72.1

Note: Deflection under the design load before heating was 0.10-0.15 in.

In the final phase of the investigation four beams of Type A of 4/5 scale with secondary reinforcement were made each having prestressing steel supplied by a different British wire manufacturer, the wires being designated as E, F, G, and H. (Wire of Make H was used in the other beams in this phase). Samples of these wires were submitted to the National Physical Laboratory for a comprehensive investigation of their properties. It was shown[6] that, while there was no significant difference between the wires with respect either to their tensile strength at normal temperature or to the temperatures at which rupture occurred when they were heated while loaded to 50 percent of their tensile strengths at room temperature, Wire E showed considerably less creep in the range of temperature from 302 to 482 F. This suggested that for beams which are subjected to fires of shorter duration than would cause collapse, a beam stressed with Wire E should show a better recovery and retention of elastic properties at normal temperature than beams stressed with the other makes of wire.

The four beams in this series of tests were subjected to the standard furnace heating for a period sufficient to raise the final temperature of the wires to predetermined levels. After the beams had cooled, they were loaded to failure in the frame through which the load was applied in the fire tests.

The results of the tests, which are summarized in Table 6, show that, of the three beams with cables heated to similar temperatures of just under 392 F the beam stressed with Wire E showed the least damage in the reloading test. The differences in performance between these three beams were not substantial and were not as great as was expected from the tests on the wires. It, therefore, appears that the considerable increase in deflection under the working load resulting from the partial fire test was only partly due to creep of the steel and that loss of tensile resistance in the concrete contributed largely to the reduction in stiffness. The cable in the beam stressed with Wire H was more severely

heated to a mean temperature of 513 F, which, according to the results of the tests on the wire, was sufficient for complete loss of prestress; the large deflections in the reloading test on the beam stressed with this wire tend to confirm this result.

The failing loads for the beams stressed with Wires E, F, and G, shown in Table 6, are about 20 percent in excess of the calculated failing loads. Part of this excess may be due to frictional restraint at the supports in the frame of the furnace, since rollers are not used in fire testing, and part may be due to a small increase in the tensile strength of the steel as a result of heating. By comparison, the strength of the more severely heated beam stressed with Wire H has been reduced but by not more than 10 to 15 percent.

A summary of the evidence from these tests shows that where beams are exposed to fire of insufficient duration to cause collapse, the damage to the beams may be sufficient to require their replacement. The criterion of damage is of necessity the subsequent effect on deflection under working loads, and the tests have shown that, after the mean temperature of the cables has reached 356 F, this deflection has become unacceptable irrespective of the type of wire used. For the type of beam tested, this temperature was reached after heating in the furnace in the standard manner for a period of slightly less than 2 hr, and replacement of these beams would therefore become necessary after exposure to fire for a period of about half that causing collapse. If there is any doubt of the future serviceability of a prestressed concrete beam damaged by fire, measurements of deflection in a loading test within the working range of load should be obtained; if deflection is not greater than that expected, it may be assumed that ultimate strength is not impaired. Repair of damaged beams would require the replacement of damaged concrete by new concrete bonded effectively to the old, and the stressing of new cables as it would not be practicable to restress the old cables which, apart from any loss of prestress, might still be serviceable; the process and difficulty of repair however might well be more expensive than the complete replacement of the member.

Effect of other factors

The aggregates used in the concrete throughout the investigation were river sand and gravel. Experience with other natural aggregates shows that their use would not significantly change the thicknesses of concrete cover required for the different periods of fire resistance, and that secondary reinforcement would still be necessary to retain the cover in position around the cables for periods of fire resistance of 2 hr or more. A given fire resistance might be obtained with a smaller cover of concrete by using artificial aggregates which give concrete better thermal properties than those resulting from the use of natural aggregates. Further research is necessary to determine whether secondary

reinforcement is required when artificial aggregates, such as foamed slag or expanded clay or shale are used.

The effects of scale have already been considered in describing the behavior of the beams in the fire test. One feature of the tests on the largest size of beams without secondary mild steel reinforcement, which requires further mention, however, was the form of failure where the slab sheared horizontally from the beam. In larger sizes of beam, connecting steel between a precast element and the cast in situ concrete should be provided, irrespective of the shear conditions or the results of structural tests, to ensure that the heavy distortions occurring as a result of heating and drying out of the concrete do not cause separation particularly where there are discontinuities in the section.

CONCLUSIONS

The investigation described in this paper has shown that, as with other forms of construction, adequate fire resistance can be attained with prestressed concrete beams when proper attention is given to design. The results obtained have led to the following conclusions, which are primarily applicable to simply supported composite prestressed concrete beams with post-tensioned cables of hard-drawn steel wire:

1. Failure of a beam carrying its design load will occur when the mean temperature of the prestressing steel reaches about 752 F.

2. Fire resistance depends on the protection given to the steel. A fire resistance of 1 hr is obtained when the cover of concrete to the prestressing steel is 1½ in. Periods of 2 and 4 hr are obtained with covers of 2½ and 4 in., respectively, provided that the cover is retained in position with secondary reinforcement.

3. A substantial increase in the fire resistance of a beam is obtained by the application of some form of insulation to its exposed surfaces, provided that the adhesion of the insulation to the concrete is ensured for the whole of the fire period. Precast slabs of lightly reinforced vermiculite concrete used as a mold lining when casting the beam are equivalent in insulating value to about twice their thickness of concrete. Evidence from other fire tests suggests that smaller thicknesses of vermiculite gypsum plaster or sprayed asbestos, in each case applied directly to the surface of the beam, should provide similar insulation to that obtained with vermiculite concrete in the tests, being equivalent to a maximum of three times their thickness of concrete.

4. The failure of prestressed concrete beams with secondary reinforcement is gradual, and is accompanied by large deflections which give ample warning of collapse.

5. If the temperature of the steel in a prestressed concrete beam exposed to fire has exceeded about 356 F, the reduction in the stiffness of the beam is likely to be severe and will normally necessitate replacement of the structural member.

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Title No. 57-63

Concrete for the Mammoth Pool Power Tunnel

By NEVILLE S. LONG and THOMAS W. HOWELL

Construction of the Mammoth Pool Power Tunnel provided an excellent opportunity for obtaining concrete at a minimum cost with maximum use of local materials.

The combination of an owner operated aggregate plant and a cooperative contractor made possible the achievement of minimum cost concrete. Satisfactory 2000 psi concrete in the tunnel invert was produced from crushed aggregate using as little as 325 lb of cement per cu yd.

■ PLANS OF THE MAMMOTH POOL POWER TUNNEL called for a fully paved concrete invert throughout its 8 miles of length, as well as concrete lining wherever required by ground conditions. Concrete on the project also included surge shaft and gate shaft lining, transition structures, and backfill concrete behind steel lined sections at the outlet portal and at the Shakeflat creek crossing. All of the aggregate used in manufacturing the tunnel concrete was produced from the tunnel muck in a crushing and screening plant owned by the Southern California Edison Co., and operated for them by the Bechtel Corp., who were constructing the dam and powerhouse portion of the project.

The Utah Construction Co., who held the contract for the tunnel construction, utilized two portable batch plants which were located at three different sites (see Fig. 1). Concrete was batched into rail-mounted transit mixers, hauled into the tunnel and discharged, either directly, into a pumping machine, or onto a wet belt. In paving the tunnel invert, 3 in. aggregate was successfully used in a 6 in. thick pavement which was machine trowel finished. Concrete operations were generally on a 24-hr basis and paving rates of up to 300 ft per shift of invert concrete were often achieved.

Springs and water drips entering the tunnel at various locations around the periphery presented particular problems during the concreting operations.

Close coordination between the construction, construction engineering, and design forces resulted in substantial savings in cement without significant reduction in quality of the finished product or the creation of placement problems for the contractor.

ACI member **Neville S. Long** is a senior construction engineer, Southern California Edison Co., Los Angeles. He was the company's resident engineer for construction of the Mammoth Pool hydroelectric project. He has been resident engineer on two other major California dams and has had 14 years experience in the construction of dams, and hydro and steam generating plants.

ACI member **Thomas W. Howell** is a senior field engineer, Bechtel Corporation, San Francisco. He was responsible for mix proportioning and control of concrete placed in the Mammoth Pool tunnel and powerhouse. He has had 20 years experience as a concrete technician on major western water development projects including Hoover Dam.

Finishing problems presented by the curved invert make use of a machine trowel finish on a curved invert a questionable item for any future work.

Inspection of the tunnel after initial filling revealed only one minor problem: In a few areas where high subsurface spring flow exceeded the capacity of the drain holes (2-in. holes on nominal 25-ft spacing, plus additional holes as directed) the pavement had heaved and water was flowing between the pavement and the sidewall of the tunnel. This indicated that nothing was gained from efforts to achieve good bond in this area. In fact, no heaving occurred in areas of high subsurface flow where bond between the invert and the sidewall had been incidentally prevented because sheet metal pans had been used to keep water off the invert during placing and finishing operations. The opening between the pan and the sidewall carried water running down the sidewall into drains under the invert. It now acts as a relief drain when the tunnel is emptied.

GENERAL DESCRIPTION OF WORK

The Mammoth Pool Power Tunnel is an integral part of the Southern California Edison Company's Big Creek hydroelectric power project, which develops the hydroelectric potential of the upper San Joaquin river. The Mammoth Pool project is located approximately 80 miles northeast of Fresno, Calif., in the Sierra Nevada Mountains. The general

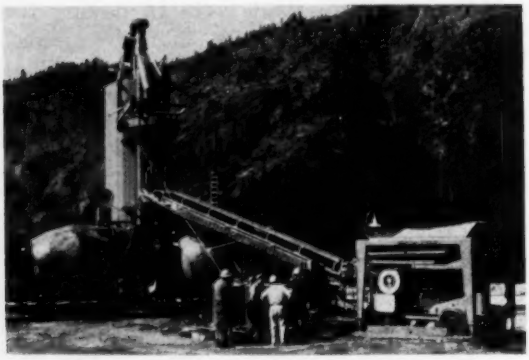

Fig. 1—Portable batch plant at Shakeflat

topography of the project area is typically Sierran with steep walled canyons, swift mountain streams, and moderately wooded slopes alternating with barren granite domes.

The project consists of the Mammoth Pool Dam, a rolled fill structure with a total height of 469 ft and a height above streambed of approximately 330 ft. The dam will create a reservoir 8 miles long with a gross capacity of 123,000 acre-ft. The Mammoth Pool Power Tunnel is a 20 ft diameter horseshoe tunnel (see Fig. 2) running approximately 8 miles from the Mammoth Pool Lake along the north bank of the San Joaquin River. At the downstream end of the tunnel, water is discharged into a steel penstock approximately 2600 ft long which carries the water down to a two-unit powerhouse with a capacity of approximately 150,000 kilowatts.

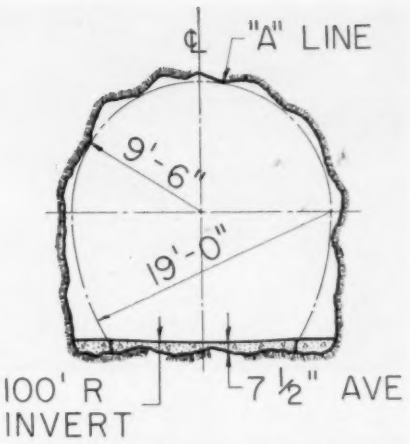

Fig. 2—Typical horseshoe tunnel section

Work on the Power Tunnel commenced in June 1958 and it was in use by December 1959.

Tunnel driving was carried out on five different faces and concrete placement operations followed this same pattern. Concrete placement was carried out from the outlet portal, from the Rock Creek adit, both upstream and downstream, and from the Shakeflat crossing, both upstream and downstream. The Shakeflat crossing was approximately 1 mile from the tunnel intake and the Rock Creek adit was approximately at the center of the tunnel.

AGGREGATE AND CONCRETE PLANT

To provide aggregate for the Power Tunnel concrete, the powerhouse concrete, penstock concrete, miscellaneous concrete in the dam, and screened gravel for special drain sections in the dam, a movable crushing and screening plant was used. The plant was rated at 125 tons per hr and was semiportable in nature (see Fig. 3). The plant was originally erected at the outlet portal where tunnel muck was used in producing aggregate for use at that end of the project. It was subsequently moved to the Shakeflat area where it was used to crush and process tunnel muck aggregate for the upstream portion of the project.

Tunnel muck was hauled out of the tunnel in muck cars and then stored. The aggregate was transported to the crushing plant which contained a standard jaw crusher, a cone crusher, primary and secondary

Fig. 3—Aggregate plant at Shakeflat

screens, and a double screw classifier. The plant was capable of producing 3, 1½, and ¾ in. nominal sizes of aggregate and sand.

The temperature and moisture of the coarse aggregates were partially controlled during the hot summer months by water sprinklers placed between the stockpiles.

MIXING PLANT

Two 30 cu yd per hr portable batch plants were used. One plant was used at Shakeflat and the outlet with the other being used at Rock Creek.

MIXERS

Transit mixers, of 6½ cu yd capacity, mounted on narrow gage railroad cars and agitated by electric motors, were used for mixing and transporting the concrete. Electric power for operation of the transit mixers was available at the mixing plant and at the placing site. Mixer motors were operated on 440 v power which was provided by plug type connectors. During the up to 2-mile trip to the placing site, no agitation could be provided. However, the required mixing at the site eliminated any possible trouble from this source. The contractor found that care had to be used in starting the mixers particularly if they were kept waiting because the concrete tended to hang in the drum during the first revolution and too rapid starting could cause the car to tip over.

PLACING EQUIPMENT

All of the concrete placed in lined sections of the tunnel was placed by pumping through 8 in. slick lines. Concrete placement in the surge shaft and gate shaft was by crane handled concrete buckets discharging into job constructed slip forms. The paved invert and gate shaft transition section is shown in Fig. 4. Placement of concrete in the invert was accomplished in two stages. Curbs were first constructed with

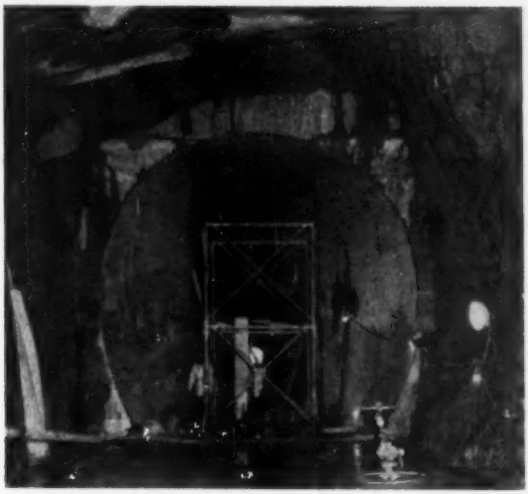

Fig. 4 — Paved invert and gate shaft transition section

Fig. 5—Placing concrete in curbs

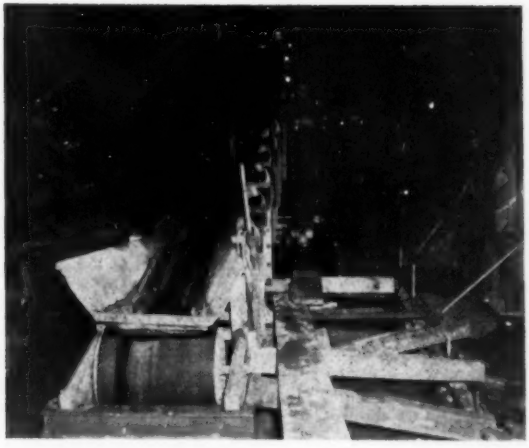

Fig. 6—Wet belt in tunnel

an opening of 13.5 ft between each curb. Placement into the curbs was directly from the concrete mixer cars (see Fig. 5), while placement of the concrete in the curved central portion of the invert was accomplished by a wet belt. The use of a wet belt was necessary to provide a working space for the track gang to remove the track between the mixer car which had to use the track and the concrete placing operation. The wet belt was job constructed and ran on solid rubber-tired wheels which traveled on the previously placed curbs (see Fig. 6). It was pulled by the locomotive which brought in the transit mix cars. Spring guides rode on the inside of the two curbs and helped to keep the wet belt lined up properly. Concrete was discharged from the transit mixer to the wet belt, carried over the track removal gang and dumped into a swinging hopper which was used to distribute the concrete across the placement area.

CONCRETE FINISHING EQUIPMENT

All concrete was vibrated with pneumatically operated internal vibrators except that form vibrators were also used on the steel forms to assist in vibration of the tunnel lining concrete. Rough finishing of the tunnel invert was accomplished by a job constructed screed. Several modifications of this screed were made during the construction and in final form it was approximately 12 ft long, 14 ft 6 in. wide, 8½ in. high and constructed to give the required 100 ft radius. The screed was mounted on wheels which rode along the previously constructed curbs. It had a strike off plate extending 3 ft to the front and two I-beams extending to the rear of the screed. These I-beams supported a working platform for two finishers. Four 3 in. immersion type air vibrators were mounted at the front of the screed. The screed had two openings located at the front and approximately 1½ ft in from each side which were built up in the form of hoppers so that a surcharge of wet concrete could be continually maintained adjacent to the curbs. The screed was pulled by two 4000 lb air tuggers which were mounted on it and which pulled against steel bars inserted in holes drilled in the sides of the tunnel. Two manually operated vibrators were in constant use by the men working on and in front of the screed.

CONCRETE MATERIALS

The aggregate from the outlet of the power tunnel was predominately a dark blue-grey, medium grain, fresh quartz dioritic granite. This granite is hard and dense, showing little secondary breakdown on crushing, and thus providing a minimum amount of sand. Small amounts of free feldspar and quartz were present. These came from the aplite and quartz dikes which were intersected by the tunnel. The aggregate crushed at Shakeflat was composed of metamorphic quartz

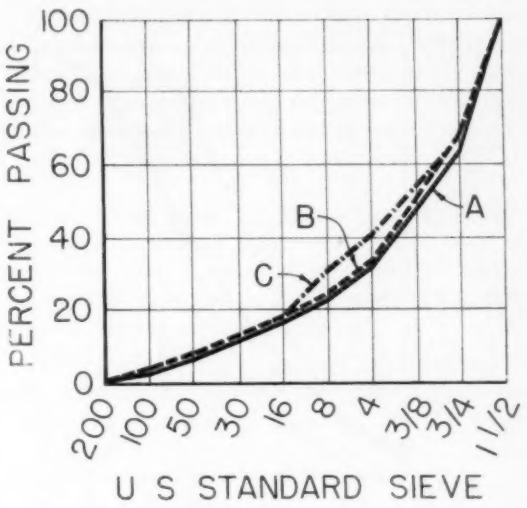

Fig. 7 — Combined aggregate gradation curve showing variation in aggregate gradation experienced in normal concrete placing operations

Line A—Design gradation
Line B—Actual gradation; sample from concrete
Line C—Actual gradation; sample taken one hour later than Sample B

ites, schists, and igneous granites and granodiorites. The breakdown of the materials on crushing at Shakeflat was greater thus producing a higher percentage of sand at this location. The average specific gravity of the outlet aggregate was 2.77 and the average specific gravity of the Shakeflat aggregate was 2.60. Percentages of 1½, ¾, and fines at the outlet were 45, 35 and 20 percent. At Shakeflat they were 18, 20, and 62 percent. Typical gradation curves are shown in Fig. 7 to 9.

Experience at other projects in the Sierra Nevadas has shown that sometimes it is necessary to use a rodmill to produce satisfactory sand for concrete from a crushing operation. The capital cost invested in the crusher plant was quite high for the amount of aggregate required and to keep costs low, it was decided to purchase the plant without the rodmill and to allow some deviation from specifications in the sand should the plant produce a sand which was reasonably satisfactory for concrete manufacture. As can be seen from the accompanying curves, the sand which was produced as a by-product of the tunnel blasting and crushing operations with just an ordinary screw classifier had reasonably good gradation, and while some slight savings in cement used could probably have been achieved with a more perfect sand gradation, these savings certainly would not have paid for the addition of a rodmill to the plant. Plans did call for the purchase of a rodmill should it have been necessary.

Cement

Type II low alkali cement was used in all power tunnel concrete. A manufacturer's certification was delivered with each shipment of cement and in addition random samples were sent to a commercial

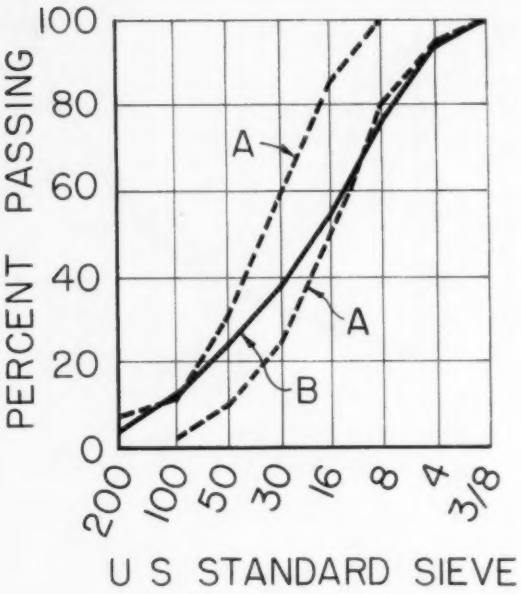

Fig. 8 — Shakeflat fine aggregate gradation tests

Lines A—Specification limits
Line B—Average of all tests at this location

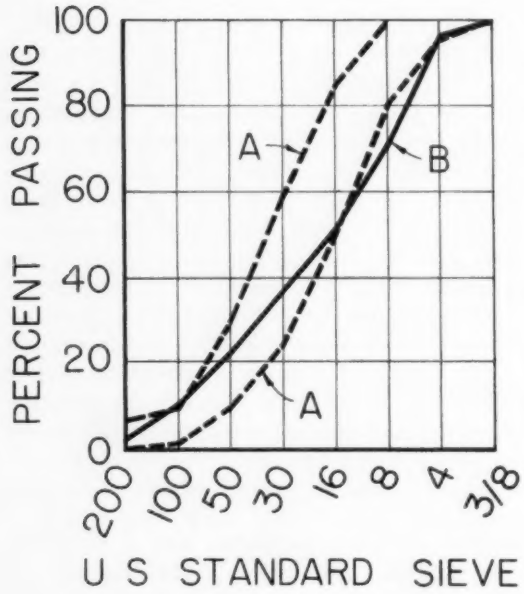

Fig. 9 — Outlet fine aggregate gradation tests

Lines A—Specification limits
Line B—Average of all tests at this location

laboratory for chemical analysis. The cement was delivered to the job site in truck and trailer rigs carrying an average load of 150 barrels. With only 250 barrels of storage capacity with each site, there was considerable standby time of the hauling equipment and close scheduling was required on the part of the contractor by this situation.

Admixture

Two types of cement dispersing water reducing type admixtures were used. One which did not alter setting time was used at the rate of 0.20 lb for each sack of cement. The other material was used at a rate of 0.25 lb per sack of cement, and in addition to its cement dispersing properties, it delayed the time of initial set of the concrete.

Extensive field laboratory tests were made to determine whether or not the use of these admixtures would be beneficial. These tests indicated that a cost savings could be realized in addition to the more intangible advantages such as ease of placement which results from the use of a cement dispersing admixture.

Reinforcing

No reinforcing was called for or placed in the paved invert sections of the tunnel. In areas of poor rock where supports had been installed during construction and where a horseshoe shaped lined concrete section was constructed, no reinforcing was included other than the embedment of the structural steel column tunnel support (see Fig. 10). A total of 443 lineal ft of tunnel was lined in this manner. In one area where the tunnel passed through an accumulation of rock and dirt, a fully reinforced circular concrete section was constructed. Reinforcing in this section consisted of two layers of #9 bars at 4 in. on centers normal to the tunnel axis with two layers of #5 bars at 12 in. on centers parallel to the tunnel axis. This double hoop type reinforcing extends for a total distance of 156 ft. On each side of this section there are additional circular reinforced lined sections which act as transitions from the fully reinforced section to the normal unsupported tunnel sections. These are reinforced with a single layer of #8 bars at 12 in. on centers normal to the tunnel axis and a single layer of #5 bars at 18 in. on centers parallel to the tunnel axis (Fig. 10). The total length of these transition sections is 105 ft. In the surge shaft lining, no reinforcement was included other than the embedment of the structural supporting members which were placed during the sinking of the shaft. In the surge drift, an area of particularly poor rock was supported by constructing a shotcreted arch with #4 reinforcing steel embedded in the shotcrete to provide reinforcement. Reinforcing bars were placed in the transition structures as called for by normal structural design.

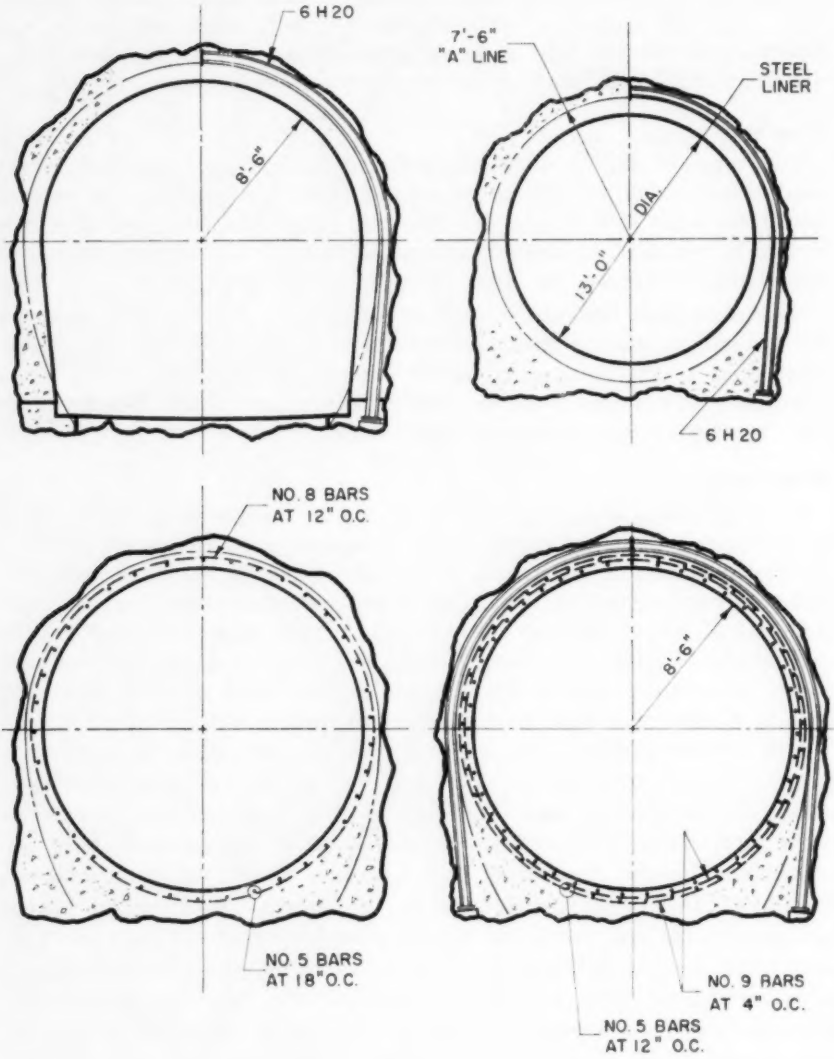

Fig. 10—Typical tunnel sections. (top left) Unsupported and supported concrete lined horseshoe. (top right) Unsupported and supported steel liner backed with concrete. (bottom left) Unsupported reinforced concrete lining. (bottom right) Supported reinforced concrete lining

Air entrainment

To achieve the 3 to 5 percent volume of air required by the specifications, 3 to 9 ounces of air-entraining agent were used per cubic yard of concrete. The final results showed that a water reduction of 8 percent was achieved on the average in the power tunnel concrete.

CONCRETE PLACEMENT

Water control

Various water control methods were used in the tunnel while placing concrete in the curbs, invert, arch, intake, gate structure, and the steel liner backfill. The methods used while placing concrete in the curbs and invert are described in detail, generally similar procedures were employed while placing concrete in the arches, liner, as backfill, and in other structures. The surge chamber was dry and consequently no water control problems were presented. Water control was divided into three phases:

1. Water control before placing
2. Water control while placing
3. Water control after placing

Curb placement

1. Before placing—Prior to placing curb concrete occasional conditions such as dead water, created by low spots in the invert grade and springs in the tunnel rib, required pumping. The tunnel invert slope of 0.002 prevented pumped water from running back into low spots if it was carried from 50 to 100 ft downstream from the area being worked on. Springs from the toe of the rib were controlled by French drains emptying into the invert.

2. Water control while placing—Control was necessary to take care of water seeps and springs originating above the curb on the tunnel rib. Initially, strips of corrugated metal were placed between the concrete and the rib allowing the water to run down the rib into a French drain and underneath the curb. This method was discontinued because it was preventing bond between the curb and the tunnel rib and at that time it was believed that this was undesirable. The second method used to control this water was the construction of drains in the surface of the curb. These were constructed by embedding 2 in. x 2 in. x 8 ft long timbers longitudinally in the fresh concrete. After the concrete took its initial set, these wooden strips were removed thus creating a longitudinal drain or gutter in the curb. This method was not as satisfactory in keeping water off the curb, however, it did serve to keep rib drainage from the surface of the central section of the invert concrete when it was being placed and finished.

3. Water control after placing—To reduce the amount of free water on the surface to be finished, corrugated steel sheets, 2 x 10 ft, were leaned against the rib to provide a roof over the curb. This roof kept drips from falling onto the curb. It was removed temporarily while the actual finishing operations were taking place.

Invert placement

1. Control prior to placing—The first invert concrete was placed between the intake portal and the Shakeflat crossing. In this area, water was dried up ahead of concrete placement by pumping the water and discharging it downstream ahead of the placing operation. This method was not entirely satisfactory because low areas had to be isolated so that the water would not run back into them and in addition there was no satisfactory way to eliminate the water that leaked into the tunnel after the concrete was placed. Downstream from the Shakeflat crossing, a 4-in. discharge line was installed approximately 4 ft above the curb and excess water was pumped into this line. As work progressed down the tunnel and the water accumulation increased, another 4-in. pipe was added and ultimately an additional 6-in. line was required to handle the flow before placing operations reached the Rock Creek adit. Two 6-in. electric pumps were used on the 6-in. line. Water was pumped into the 4-in. lines with air-driven pumps. For the first 3000 ft downstream from the Rock Creek adit, no discharge line was needed as this area of the tunnel was relatively dry. A 4-in. line was sufficient to handle water in the next 10,000 ft of the tunnel and as the water accumulation increased, it was necessary to install a 10-in. line which was sufficient to handle the water encountered in the final 6000 ft of the tunnel. Water was pumped into this pipe by two 6-in. pumps and one 8-in. pump, one 6-in. pump was located at the upper end of the pipe, the second pump being installed 2000 ft farther downstream. At the rocktrap 500 ft from the outlet, the 8-in. pump was installed. A pneumatic diaphragm pump capable of picking up as much as 40 percent solids was used in mucky areas of the invert. This pump discharged into an empty agitator which hauled the material to the outside. Hauling of this material away from the tunnel eliminated its continuing rehandling as the work progressed downstream.

The specifications permitted the contractor to use tunnel muck to fill low areas in the invert with placement of the invert concrete on this tunnel muck being permitted. The contractor's normal driving operations resulted in an over-break in the invert and with only a few minor exceptions, no final rock trimming in the invert was required. In some cases, ballasting and reballasting of the railroad track had built up the ballast above allowable grade although generally the contractor did a good job in keeping the ballast grade at or below the required grade. Thus, the invert concrete was generally placed on

several inches to a foot or two of inplace tunnel muck. The specifications did permit isolated bedrock points to intrude into the 6 in. specified invert pavement, however, this rarely happened except in the curbs adjacent to the tunnel rib.

In locations where heavy spring flow was encountered, extensive French drain systems were installed. These drains were constructed by digging a trench 6 to 8 in. below grade, placing perforated pipe in the trench and covering with 1½-in. aggregate. All these drains emptied into major sumps which were temporarily left in the invert concrete. All major sumps were left as permanent drains by installing 4 or 6-in. pipes in the finished invert. In some areas of the tunnel where the invert was heavily contaminated with oil or grease, building paper was laid over the invert just ahead of concrete placement.

2. *Control of water during placing*—Immediately prior to and as a part of the placing operation, a 2-in. pneumatic pump installed 15 ft ahead of the screed on the wet belt frame was hand-operated to pick up free water in depressed areas of the surface. This is a vacuum type pump and its use is similar to that of an ordinary vacuum cleaner. The discharge line for this pump was attached to the wet belt frame and extended ahead to discharge approximately 125 ft downstream. Another 2-in. pneumatic pump was mounted on the rear of the screed to pick up free water that accumulated immediately behind the screed. This pump also discharged downstream. Where heavy water concentrations were encountered, sumps were installed in the fresh invert concrete and additional air pumps were placed in the sumps pumping directly into the discharge lines as described above.

3. *Water control after placing*—After screeding of the invert and also while placing concrete in extremely wet areas, a canopy was used to keep water from the surface of the concrete. This canopy was 7 ft high, 14 ft wide, and 40 ft long. It was constructed with a pipe frame covered with plastic sheets and was mounted on rubber-tired wheels which permitted it to be pushed along the tunnel with the wheels running on the finished curbs. In other areas, the invert was covered with waterproof paper after it had been screeded and bull floated. The paper was removed while the invert was being machine trowel finished and replaced until the concrete had achieved its final set.

CONCRETE MIX PROPORTIONING

The construction specifications established class, maximum aggregate size, and maximum slump in Table 1.

The contractor's construction plans called for placing of all nonreinforced backfill concrete with a concrete pump and because this machine cannot pump 3-in. size aggregate, the maximum aggregate size used for unreinforced concrete backfill was 1½ in. In nearly all of the invert concrete placed from the Rock Creek and in a part of the

TABLE 1 — CONSTRUCTION SPECIFICATIONS

No.	Location	Minimum strength, psi	Maximum aggregate size, in.	Maximum slump, in.
1	Reinforced concrete lining	3000	1½	3½
2	Unreinforced concrete backfill	2000	3	3
3	Adit plug	3000	1½	3
4	Intake portal and transition structures	3000	1½	3
5	Gate structure at tunnel enlargement	3000	1½	3
6	Reinforced concrete at top of gate shaft	3000	1½	3
7	Invert lining	2000	3	2
8	Miscellaneous structures	3000	1½	3

invert concrete placed from Shakeflat 3-in. aggregate was used. However, no 3-in. aggregate was used in concrete placed from the outlet portal because none was readily available at this location.

The specification slump in all of the concrete that was pumped had to be increased because of job conditions. The crushed aggregate mixes could not be pumped at a 3 or 3½-in. slump and a slump from 4 to 6 in. was permitted in pumped concrete. With this exception, specification slumps were generally obtained throughout the project.

CONTROL OF PLACING OPERATIONS

Tests of fresh concrete at the batch plant

Concrete was discharged out of the mixer into a wheelbarrow and with this sample, a series of tests were conducted. When 3-in. aggregate was being used, the mix was screened on a 1½-in. screen to remove all oversize particles; otherwise the sample was used without screening. Slump, unit weight, and entrained air tests were taken from the sample and two to six standard cylinders were cast. The air and unit weight tests were taken by using a 0.25 cu ft air meter. The air content was regularly checked by the pressure method and was periodically checked by the gravimetric method. The volume of air in the paste or mortar usually checked closely.

Tests of fresh concrete at the placing site

As the concrete was discharged from the mixer onto the wet belt or into the forms, a sample was taken from the same mixer that a sample had been taken from at the batch plant. The confined area at the placing site coupled with the relatively high speed placing operations always created considerable congestion at the placing site as well as putting a premium on available space. For this reason and because of the difficulty in getting the testing equipment in and out of the tunnel, only a few check tests were taken at the placing site. Enough tests were taken at the site to determine that there was an

TABLE 2 — SUMMARY OF TESTS OF FRESH AND HARDENED CONCRETE

Structure	Total concrete, cu yd	No. of tests	Cement per cu yd, barrels	Average values				
				Air, percent	Slump, in.	Unit weight, lb per cu ft	Compressive strength	
							7 days	28 days
Invert lining, 3 in.	15,800	92	0.86 to 1.04	4.2	2.87	145.2	1620	2062
Invert lining, 1½ in.		74	1.06 to 1.20	4.3	3.29	147.4	1390	2350
Unreinforced concrete backfill	10,360	38	1.06 to 1.41	4.2	4.88	148.2	1550	2800
Reinforced concrete lining		10	1.41 to 1.70	4.5	5.82	145.7	—	3260
Gate shaft, surge shaft, arch, transition section and miscellaneous concrete	2270	26	1.49	4.2	4.08	147.7	2370	3580
		17	1.41	5.2	5.15	144.6	1760	2810
Job average			1.16	4.4	4.35	146.5	1740	2810
Total	30,972	257	36,100 bbl					

average slump loss of ½ in. between the batch plant and the placing site. Compression tests made on cylinders taken at the site averaged higher than those taken at the batch plant. Job practice called for all cylinders being left undisturbed for 24 hr after casting and this resulted in some cylinder loss of the cylinders taken at the site before they could be removed from the tunnel.

Tests of hardened concrete

The standard 6 x 12-in. cylinders were cast in wax cardboard molds and when cast at the batch plant, were stored in a construction shed. Samples were taken from each 250 cu yd of one mix and from mixes of lesser amounts of concrete. Generally, four cylinders were cast for each test. One pair of cylinders was broken at 7 days and the other pair at 28 days, except that a few 90-day, 180-day, and 1-year cylinders were also cast. As the concrete production increased, with two batch plants working three shifts, the curing room was unable to accommodate the large number of cylinders and as the mixes had become reasonably standardized the breaking of 7-day cylinders was discontinued for the remainder of the job. Cylinders were broken by a motor-driven hydraulic compression machine of 300,000 lb capacity. A summary of tests on fresh and hardened concrete is shown in Table 2.

Concrete mix data

A complete list of the mixtures used is given in Table 3. Because of the aggregate gradation fluctuation and moisture fluctuation, the W/C ratio of a given mix fluctuated as much as 0.07. During the initial stages of the job, mixes were proportioned and changed frequently to try and maintain a constant W/C ratio. As the job progressed a mix

TABLE 3 — MIX PROPORTIONS USED FOR TUNNEL CONCRETE

Mix No.	Cement content, lb per cu yd	Mix parts, cement = 1.00	Specified air content, percent	Theoretical unit weight, lb per cu ft
1B-3P	640	0.52:1.00:2.09:2.66	6	148.5
2D-3P	450	0.70:1.00:2.31:2.31:2.60	5	148.7
2D-4P	450	0.70:1.00:2.44:2.38:2.38	5	148.3
2D-5P	420	0.69:1.00:3.17:2.45:2.31	5	149.6
2D-6P	400	0.63:1.00:3.35:2.60:2.42	5	149.6
2D-7P	400	0.70:1.00:2.75:2.73:3.17	5	153.3
2D-8P	475	0.61:1.00:2.77:2.15:2.01	5	150.2
2D-9P	450	0.64:1.00:2.93:2.29:2.13	5	150.0
2B-5P	560	0.52:1.00:1.84:1.91:2.00	5	150.7
2B-6P	560	0.52:1.00:2.02:1.89:1.84	5	150.7
2B-7P	560	0.52:1.00:2.30:1.84:1.61	5	150.7
2B-8P	500	0.58:1.00:2.62:2.10:1.82	5	150.4
2B-9P	530	0.55:1.00:2.45:1.96:1.72	5	150.7
2B-10P	530	0.55:1.00:2.57:2.04:1.79	5	155.9
3D-5P	390	0.70:1.00:2.09:2.18:2.27:2.18	4	150.6
3D-6P	390	0.70:1.00:2.36:2.26:2.10:2.00	4	150.6
3D-7P	376	0.72:1.00:2.71:2.37:2.18:1.81	4	150.4
3D-8P	350	0.77:1.00:2.94:2.57:2.37:1.97	4	150.7
3D-9P	325	0.77:1.00:3.23:2.83:2.62:2.15	4	151.7

Design notes:

1. Mix volume = 1 cu yd absolute volume plus specified air content.
2. Specific gravity used for cement was 3.15.
3. Specific gravity used for fine and coarse aggregate was 2.65, with the exception of Mix No. 2D-7P. The specific gravity of the fine aggregate was 2.60 and the coarse aggregate was 2.77, and for Mix No. 2B-10P the specific gravity of the fine and coarse aggregate was 2.77.
4. All mixes were proportioned with water-reducing and air-entraining admixtures.

was adopted and used until a definite trend indicated that it should be changed.

In the proportioning and control of the invert concrete, standard procedures were modified to achieve maximum economics in cement usage. The specified strength of the invert concrete was 2000 psi. The company has recently adopted the ACI criteria of proportioning structural concrete on the statistical basis of requiring that not more than 10 percent of the tests fall below the specified strength. If this criterion had been followed in the case of the power tunnel invert, the concrete would have had to be proportioned for a 3000 psi average, which would have called for approximately 430 lb of cement per cu yd. Concrete proportioned for a strength of 2000 psi without giving any consideration to the normal variation which is to be expected calls for a W/C ratio of 0.77 and a cement content of 325 lb per cu yd. It was decided to use this mix and to recognize that some tests would fall below 1500 psi. However, strength was not the prime criterion in this concrete, and it is believed that the results obtained were satisfactory. Attention is called to Table 2.

TUNNEL SHOTCRETE

The primary application of the tunnel shotcrete was in covering areas of slaking rock, and badly jointed areas which were not bad enough to require lining. Additional applications for tunnel shotcreting included transitions at the beginning and end of the concrete lined sections and at the end of the steel liner pipes. In addition, extensive shotcrete was applied in the lower regions of the surge shaft and in the surge drifts. Almost without exception, shotcrete was reinforced with wire mesh doweled into the rock.

The shotcrete mix contained 6 sacks of cement per cu yd of sand. Sand for the shotcrete was obtained from the same aggregate plant that produced all concrete aggregates for the project.

Shotcrete rebound averaged 40 to 50 percent reflecting the difficult placing conditions inherent in all underground work where the combination of shooting on side walls and roof, and in wet sections of the tunnel increased the rebound quantities above those expected under more favorable conditions.

In the wet areas of the tunnel, considerable difficulty arose in keeping the shotcrete from falling off the ribs and arch before it could take its initial set. To combat this problem, several different methods were employed. The contractor was given approval to add calcium chloride in amounts not exceeding 2 percent by weight and also to use an accelerator cement in proportions up to 25 percent of the portland cement content. Prior to placing shotcrete on a wet surface, the contractor used petroleum fired weed burners to dry the surface. The shotcrete was then applied immediately and the weed burners were again used to speed up the drying of the fresh shotcrete. It was found that the combination of an accelerator and the applied heat produced a quick enough set to give satisfactory results in most instances where shotcrete was applied in wet areas.

Over approximately 7 months 18,299 sacks of cement were used for shotcrete and 11,466 lb of 6 x 6—10/10 wire mesh was installed for reinforcing. The average 28-day breaking strength of all shotcrete cylinders cast was 5380 psi.

ORGANIZATION

The project was constructed by the Southern California Edison Co. under the general direction of W. L. Chadwick, vice-president, and R. W. Spencer, manager of engineering. Construction was the responsibility of O. N. Kulberg, chief construction engineer, H. A. Barber, project engineer, and N. S. Long, resident engineer. Tunnel design was carried out under the direction of T. M. Leps, chief civil engineer.

G. W. Saul was project manager for the Bechtel Corp. which was responsible for field engineering and inspection; H. R. Castonia, tunnel engineer, T. W. Howell, concrete engineer, and W. B. Henry, chief of surveys, were directly responsible for the tunnel work.

The Utah Construction Co., held the contract for construction of the power tunnel and the work was under the direction of Boyd C. Paulson, project manager, A. J. Simoneaux, project engineer, and P. O'Doud, concrete superintendent.

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Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1961, for publication in Part 2, December 1961 JOURNAL.

Title No. 57-64

Prestressed Concrete Beams by Electronic Computer

By CHARLES WILSON

Describes the analysis and design of prestressed concrete bridge members as performed by a small-scale electronic computer. Specifically, the member must be a simply supported longitudinal highway bridge beam carrying standard AASHO trucks. To demonstrate results of programing, an illustrative problem is presented showing input data, computational effort, and computer results.

■ PRESTRESSED CONCRETE IS A relatively recent arrival to the field of engineering and construction. Research with prestressed concrete and the resulting design codes have so far provided the engineer with only limited criteria for safe design of structures. Careful analysis of load conditions and stresses at varying times in the life of the member and attention to new design details must be given by the design engineer.

Most engineering problems are solved by mathematical computation. Engineering decisions are based on the results of these computations. The electronic computer provides an economical and speedy tool which performs these mathematical operations for the engineer.

Problem scope

To use the electronic computer in the design of prestressed concrete members, the following requirements for input and output information were established.

Input
- Bridge geometry
- Beam section data
- Concrete and steel properties
- Dead and live loads

Output
- Beam section properties
- Design moments and shears
- Prestressing data
- Web reinforcement
- Stresses at design loads
- Deflection history
- Ultimate capacity investigation

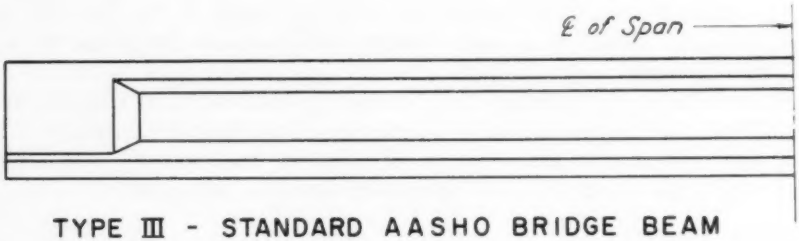
USE OF AN ELECTRONIC COMPUTER

To achieve the above requirements the first step is organizing the problem for suitable computer procedure. The investigation of a typical bridge member should indicate the order of calculations and decisions required.

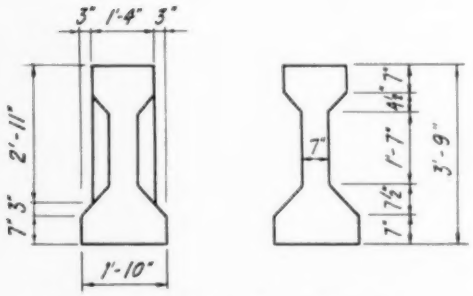
Having selected a beam, the section properties must be computed. This includes the flexural section factors for top and bottom levels of the concrete beam. Fig. 1 shows a typical prestressed concrete I-beam for a deck slab composite type structure. Because it is a standard AASHO bridge beam some section properties are already available.

If a diagonal tension study is to be made, the section factors and the first or static moments of area must be computed for as many as three additional levels on the concrete section. And as the section is to be used in conjunction with a cast-in-place concrete deck these properties must be re-evaluated for composite beam action.

From the span length, the stringer spacing, the magnitude of the dead loads, and the type of live load, the midspan moments are calculated. For a more complete investigation, conditions at the quarter points are also computed.



TYPE III - STANDARD AASHO BRIDGE BEAM



END BLOCK SECTION ℄ SECTION

Beam Data:

I (in^4)	125,390
A (in^2)	560
d (in)	45
$\bar{y}$ (in)	20.27

Bridge Data:

Span	61'-0"
Stringer Space	6'-6"
Depth of Slab	7½"

Fig. 1—Typical prestressed concrete I-beam

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Knowing section properties, design moments, and specifications governing stress conditions a satisfactory prestressing arrangement must be determined. Some of the considerations for this operation are as follows:

Critical points are the top and bottom levels of the beam at midspan and near the support.

The loading stages investigated in designing a prestressed concrete beam are, in order:

1. At application of initial prestressing
2. Prior to erection but after most time dependent losses of prestress have occurred
3. At completion of construction when, if beam is composite, it is supporting full dead and superimposed dead load
4. Under full design load
5. At ultimate load

The stresses at the critical points must satisfy specifications at each of the time intervals investigated. This will result in as many as 16 stress conditions which must be satisfied simultaneously.

There are two, perhaps three, unknowns; the magnitude of the prestressing force, its location or eccentricity at midspan, and, if post-tensioning is to be considered, the location of the prestressing force at the support. Therefore the method of prestressing if not determined prior to start of computations will depend on numerical evaluations of stress conditions. Supposing a pretensioned system to be most economical, conditions at the supports may become critical. This necessitates the use of a partially or a totally post-tensioned system.

From these considerations the problem is resolved to selecting the critical stress equations. When these equations are solved simultaneously, values assigned to the unknowns must be acceptable to all stress conditions.

This, needless to say, is largely a computational effort and ideally suited to the numerical comparisons of machine logic.

Having selected the prestressing method and steel requirements the remaining computations follow in order.

Ultimate flexural capacity for the concrete and steel sections must be checked. Both AASHO and BPR have requirements for, and methods of computing, the ultimate moment capacity of a section.

Camber must be determined for several intervals in the life of a beam. The predicted positions may be used to roughly check the manu-

facture, to specify camber, to guide construction, and to check that the permissible live load deflection ratio is not exceeded. Concrete strain as a function of both load and time must be considered in camber computations.

In prestressed concrete work, economy dictates that beam members have heavy flanges and thin webs. For design loads the assumption is that concrete has not cracked due to principal tensile stresses. If however, computations show these stresses to be severe, steel must be provided to strengthen the web. To gain sufficient knowledge of diagonal tension conditions at design load requires tedious and repetitive type calculations as several locations must be investigated. For ultimate loading the assumption is that inclined or diagonal cracking has penetrated through most of the member. The web steel must then be designed so that the shear at this load is transmitted across the crack in the beam.

Therefore to analyze web action and to design steel reinforcement, design load and ultimate load moments and shears must be calculated for several points and then evaluated with respect to the section. These evaluations, principal stresses and shear failure theory, have numerical procedures and codes specifying acceptable values.

Briefly summarizing, the design of a prestressed concrete beam can be subdivided into smaller phases each of which requires a set of numerical computations and decisions. However, these phases are not independent but follow in order and each requires information developed in prior computations.

PROGRAM REQUIREMENTS

The computer to be used, an IBM 610, had not been programed for complete design problems prior to this time, and the initial trend was toward analysis rather than design. On study it became apparent that by using the logical functions of the computer along with criteria obtained from codes and standards, the engineer could be relieved of the task of transferring data to and from the computer. Thus the problem could be suited to complete analysis and design rather than to a series of small programs with intermediate judgments by non-computer personnel. This results in a saving of engineering and computing time.

To the full extent of the operations already discussed, the program was to process the following variations of prestressed concrete beam design.

1. A composite I-beam slab type of structure or a box girder without composite slab action
2. A pretensioned or post-tensioned system of prestressing, or combination of the two methods. Further to select the method applicable in a given situation if instructed

3. A design within the requirements of both AASHO specifications and BPR criteria for prestressed concrete

This program, then, investigates the two common methods of prestressing. The first is the pretensioned system with all prestressing steel tensioned horizontally and the concrete cast around the tensioned strands. Prestressing is accomplished by releasing the tension on the strands after the concrete has strengthened. In the second method, post-tensioning, the concrete beam is cast with one or more hollow ducts which are usually draped in the form of a parabolic curve with the vertex at midspan. When the concrete has strengthened the prestressing steel is fitted through the ducts and tensioned.

In the remaining discussion the pretensioned system shall be that with straight and horizontal prestressing strands, and the post-tensioned system that with parabolically draped prestressing steel.

Two classes of criteria were developed. First were the accepted codes and standards which are subject to minor deviations from project to project. This information is published so that the engineer knows the specifications determining his design. If changes are required they may be accomplished by slight alterations of the program. This information can be found directly in the AASHO specifications and BPR criteria. The second class of information is the specific data for the particular beam to be designed.

ILLUSTRATIVE PROBLEM

To demonstrate the final results of the programing efforts, the beam in Fig. 1 will be described with respect to input data, computational effort, and computer results.

Input data

Bridge data—With the bridge data in Table 1 the program is capable of computing any of the beam sections found in Fig. 2. For Sections 3 and 4, the program stops after the flexural study. If a beam section other than Section 1 is to investigated, the input items 4 through 7 are omitted.

As the problem to be illustrated is a Section 1 type, the remaining data will concern only this section. For the other sections the data is less involved and the solutions less complex.

Beam data—The information in Table 2 defines the beam section. As this is a standard AASHO section, the code so indicates. If a nonstandard section were to be investigated, the code entry would be 1.0— and no entries are required for the computed section properties. The data format as shown is applicable to only the section in Fig. 3. The last two entries are the compressive strength of the concrete at time of prestressed application and the final or 28-day strength, respectively.

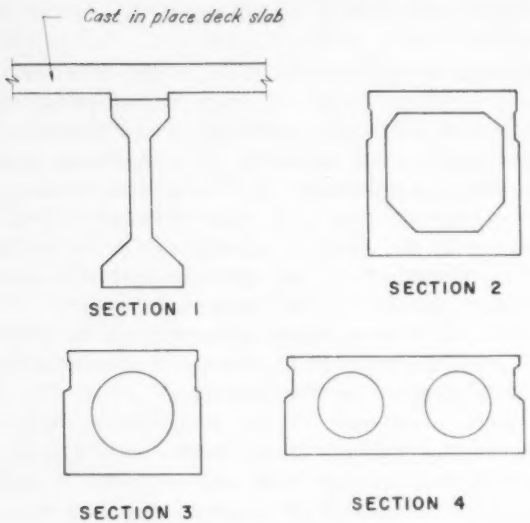

Fig. 2 — Variety of beam sections within the scope of the computer program

Prestressing data—Prestressing data are shown in Table 3. The first data entry indicates the method of prestressing desired. The 0.0+ means to try a pretensioned system, and only if that fails try a post-tensioned system. For an exclusively pretensioned system the system code would be 1.0+, and for a post-tensioned system 1.0—.

Fig. 4 explains the k entries. The ideal requirements from the viewpoint of stress satisfaction may result in a prestress force and eccentricity which cannot physically be placed in the concrete section. To insure against this occurrence, limits are placed on the steel layout.

TABLE 1 — BRIDGE DATA

1. Beam section No.	1.0+
2. Span length	61.0+ ft
3. Stringer spacing	6.5+ ft
4. Depth of slab	7.5+ in.
5. Haunch depth	1.0+ in.
6. f_c', slab concrete	3.0+ kips per sq in.
7. Fascia overhang	0.0+ ft

TABLE 2 — BEAM DATA

1. Standard section	1.0+
2. Moment of inertia, I	125390.0+ in.^{4}
3. Area, A	560.0+ sq in.
4. $\bar{y}$	20.27+ in.
5. d	45.0+ in.
6. t	7.0+ in.
7. b_1	16.0+ in.
8. c_1	7.0+ in.
9. d_1	4.5+ in.
10. b_2	22.0+ in.
11. c_2	7.0+ in.
12. d_2	7.5+ in.
13. f'_{ci}	4.0+ kips per sq in.
14. f_c'	5.0+ kips per sq in.

TABLE 3 — PRESTRESSING DATA

1. System code	0.0+
Pretensioning data	
2. f_s'	240.0+ kips per sq in.
3. f_{sy}	204.0+ kips per sq in.
4. A_s per strand	0.1089+ sq in.
5. k_1	203.0+
6. k_2	18.47+
7. k_3	1200.0+
8. k_4	12.37+
Post-tensioning data	
9. f_s'	220.0+ kips per sq in.
10. f_{sy}	187.0+ kips per sq in.
11. A_s per strand	0.1089+ sq in.
12. k_1	203.0+
13. k_2	18.47+
14. k_3	1200.0+
15. k_4	12.37+
16. u	0.25+
17. k	0.0015+

Fig. 4 shows the relationship of the sum of the strands times their distance from the centroid versus the eccentricity of the group of strands. These curves shown for the four standard AASHO sections are close to parabolic. By defining two points on the curve, the maximum capacity of the section for a given eccentricity requirement is determined.

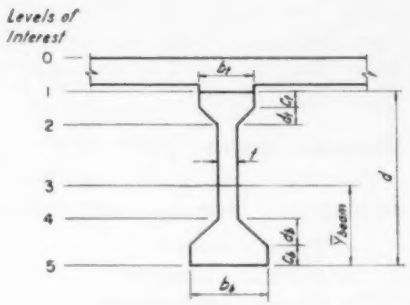

Fig. 3 — Typical prestressed I-beam, composite slab arrangement

Before a prestressing arrangement is judged final by the computer it must pass this last test. If the value of eccentricity for a given requirement of prestress force is too great, it is reduced and the value of the prestress force is increased until the curve in Fig. 4 is satisfied. This results in a prestressing arrangement which can be accommodated by the section.

If the beam is post-tensioned losses occur in prestressing force due to angle change in draped cables and wobble in the ducts. This is accounted for by the selection of μ and k for the materials involved.

Web and shear data—Items 1 and 2 of Table 4 are design stress and yield stress of the web steel. The third item is the ultimate bond capacity to be used in the design of the shear connection in case of composite action.

Load data—Data given in Table 5 are the uniform loads including dead load (excluding only the beam weight), the superimposed dead load, and the pedestrian live load.

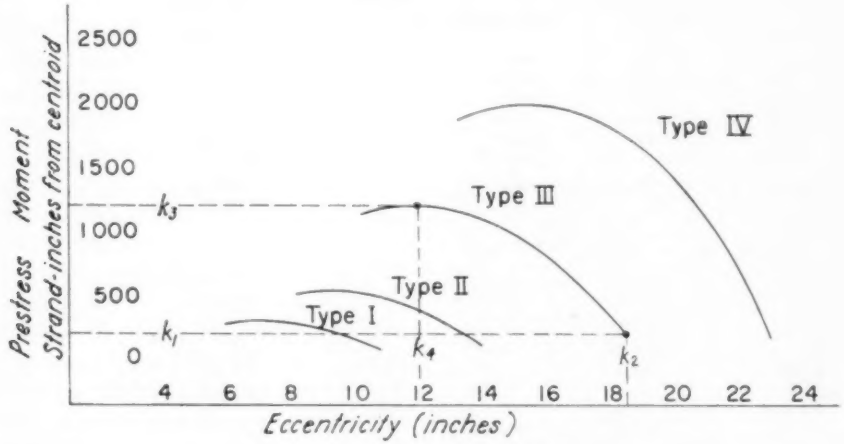

Fig. 4—Steel capacity of standard AASHO bridge beams

The live load is the H20-S16-44 truck. The a is the wheel distribution factor as found in the AASHO specifications. If live loadings other than H20-S16-44 are used, the program must be altered to accommodate them.

Computer output

The following discussion describing the computer results is subdivided into five separate studies:

1. Flexural study
2. Deflection patterns
3. Diagonal tension study
4. Ultimate shear design
5. Composite shear connection

These phases are typical for the composite type structure under consideration. So that the description of the program is more complete, beam designs for both pretensioned and post-tensioned systems will be illustrated.

TABLE 4 — WEB AND SHEAR DATA

1. f_s	18.0+ kips per sq in.
2. f_c'	30.0+ kips per sq in.
3. v_a	0.150+ kips per sq in.

TABLE 5 — LOAD DATA

1. W_{DL}	0.779+ kips per ft
2. W_{SDL}	0.363+ kips per ft
3. W_{LLP}	0.00+ kips per ft
4. a	1.3+

TABLE 6 — DESIGN MOMENTS

Impact		0.2688+
Moments	Midspan, kip-ft	Quarter span, kip-ft
DLG	271.31+	203.48+
DLS	362.33+	271.74+
SDL	168.84+	126.63+
LLP	0.00+	0.00+
LL + I	679.90+	540.61+

TABLE 7 — SECTION PROPERTIES

$W_{eff.}$	78.00+ in.
A	585.00+ sq in.
Modular ratio	0.600+

Section properties	Beam	Composite Beam
I, in.^{4}	125390.00+	314548.46+
A_t, sq in.	560.00+	911.00+
d, in.	45.00+	53.50+
$\bar{y}$, in.	20.27+	31.62+
S_t, in.^{3}	5070.35+	23523.57+
S_b, in.^{3}	6185.98+	9945.13+
S_b, in.^{3}		14381.57+

Flexural study—Flexural study results are shown in Table 6. Dead load computations separate the effects of the girder weight, the slab and diaphragm loading, and the superimposed dead load. This is necessary because of the time intervals on loading. The live load moments are produced by an H20-S16-44 AASHO truck load. For spans less than 37.5 ft modified military loading is used.

The data in Table 7 show the computed section factors for both beam and composite beam for the levels which will control the prestressing steel computations.

The cast-in-place deck slab becomes part of the top flange of the beam after it attains full strength. The structurally effective depth of the slab has been set by input data and its effective width is determined from AASHO specifications. To account for the different moduli of elasticity for deck and beam concrete, the resulting areas is multi-

plied by the modular ratio. Here the assumption is that the modulus of elasticity varies directly with the ultimate strength of the concrete.

Selection of the type of prestressing system is to be determined as indicated by input data. The computer first tried a pretensioned system as the most economical choice. The stress equations used to select the prestress force and the eccentricity are:

1. (a) Support, (b) top level, and (c) Initial prestress

$$\left(\frac{F}{A} - \frac{Fe}{S_t}\right)\frac{F_o}{F} = f$$

2. (a) Midspan, (b) bottom level, and (c) design load

$$\frac{F}{A} + \frac{Fe}{S_b} - K = 0$$

where

e = eccentricity
F_o = initial prestress force
F = final prestress force
$f = 6\sqrt{f'_{ci}} = 0.3794$ kips per sq in. (the allowable tension stress)
K = the stress effect on the bottom level, midspan, for all design loads other than prestressing

$$\frac{F_o}{F} = \frac{f_s + \text{losses}}{f_s}$$

Solution of these two equations yields values for F and e. At this point, two stress conditions are satisfied. Checking the remaining stress conditions, the computer finds the prestress values are not suitable and cannot be adjusted to satisfy all conditions. After these tests have shown that pretensioning is inadequate, the pretensioned system is

TABLE 8 — PRESTRESSING STEEL AND FLEXURAL STRESSES FOR PRETENSIONED SYSTEM

Prestressing steel

Prestress force, kips		Eccentricity, in.	A_s, sq in.	Stress, ksi	
Final	Initial			Final	Initial
1+ 617.68+	2.84629476872937+ 780.22+	11.52+	4.64+	133.00+	168.00+

Stress values, ksi

Support		Quarter span		Midspan	
Top	Bottom	Top	Bottom	Top	Bottom
0.3794— 0.3004— 0.3004— 0.3004—	2.8462+ 2.2533+ 2.2533+ 2.2533+	0.1021+ 0.1811+ 0.8243+ 1.1647+	2.4515+ 1.8585+ 1.3314+ 0.5263+	0.2626+ 0.3417+ 1.1992+ 1.6322+ 0.7081+	2.3199+ 1.7269+ 1.0241+ 0.0000+

Ultimate moment capacity, kip-ft

	AASHO	BPR
Required	2903.50+	2964.80+
Section capacity	3697.42+	3740.93+

rejected and the post-tensioned system adopted. Table 8 is the resulting design if the pretensioning method were specified as input. The computer after testing and judging the stress conditions indicates failure by a "flag" such as the 15 digit printout in Table 8, then stops. If the operator chooses to continue, he may do so. Table 8 contains the results of an unsuitable pretensioned design of the described beam. The flag indicates the stress on the bottom level at the support during the prestressing is 2.846 kips per sq in. This is above the allowable of 2.400 kips per sq in. as prescribed by specifications. Using the prestressing arrangements as shown under "Prestressing steel" yields the stresses shown under "Stress values." Note the bottom level of concrete at the support is overstressed throughout the life of the beam. Because of this extra critical condition a pretensioned system is not possible.

Table 9 is the resulting design after the computer, having disregarded the pretensioned system, adopted the post-tensioned, draped cable system. This serves to eliminate the critical conditions at the support, and evaluates prestressing requirements on midspan conditions only. Inspection of the stress values in Table 9 will reveal zero values at midspan.

1. At midspan, top level, for final prestress

$$\frac{F}{A} - \frac{Fe}{S_t} + K_t = 0$$

2. At midspan, bottom level, for design load

$$\frac{F}{A} + \frac{Fe}{S_b} - K_b = 0$$

where

e = eccentricity
F = final prestress force
K_t = stress effect on top level midspan for dead load of the girder
K_b = stress effect on the bottom level, midspan for all loads other than prestressing

Solutions of these two equations will yield the midspan values for F and e shown in Table 9. The stress values are indicated for initial prestress, final prestress, design dead load, and full design load. For the post-tensioned system selected, the critical stress conditions were at top level for final prestress and bottom level for full design load. Note that all other stress conditions are satisfied, therefore the absence of overstress flags.

The variation of prestressing force due to friction losses, and the change in eccentricity from midspan to support is shown under "Prestressing steel." As a manufacturer of prestressed concrete beams may wish to achieve the required prestress by a combination of pretensioning and post-tensioning, limits to the minimum and maximum amounts

of pretensioning are supplied. If the beam is to be lifted from the casting bed before post-tensioning operations, a certain amount of prestress is necessary before removal. This is the minimum. From Table 8 it is obvious that only a portion of the prestress can be supplied from pretensioning. As the amount and location of this auxiliary pretensioning is not known in the design phase, limits can only be set in terms of F and Fe. If these limits are exceeded the concrete will be overstressed prior to the time of final post-tensioning.

Stress values in Table 9 show the stress conditions produced by passing the centroid of the prestressing steel through the centroid of the concrete section at the support with a parabolically curved cable.

It must be noted that though stress conditions at design dead load are not critical with respect to allowable stresses, investigation is warranted on the basis of permanency of loading. Stress effects for the composite interaction of two materials, each of which is subject to creep, has not yet been defined. This program assumes no time dependent stress transfer due to dead loads. The stresses in the beam under dead load are shown and should be carefully evaluated by the design engineer.

Ultimate load investigation is required by both AASHO and BPR. The section now designed for moments due to working loads is investi-

TABLE 9 — PRESTRESSING STEEL AND FLEXURAL STRESSES FOR POST-TENSIONED SYSTEM

Prestressing steel

Prestress force, kips		Eccentricity, in.	A_s, sq in.	Stress, ksi	
Final	Initial			Final	Initial
531.48+	634.48+	15.18+			
552.93+	655.93+	11.38+			
575.10+	678.10+	0.00+	4.45+	129.00+	152.10+

Pretensioning limits

	$k_1 \times F$		$k_2 \times Fe_{centerline}$		k_3
Minimum	0.00178+	+	0.000161+	>	0.5263+
Maximum	0.00178+	+	0.000161+	<	2.0103+
	0.00178+	−	0.000197+	>	0.3178−

Stress values, ksi

Support		Quarter span		Midspan	
Top	Bottom	Top	Bottom	Top	Bottom
1.2109+	1.2109+	0.1800+	1.9837+	0.1244−	2.1636+
1.0269+	1.0269+	0.2274+	1.6102+	0.0000+	1.7269+
1.0269+	1.0269+	0.8705+	1.0831+	0.8575+	1.0241+
1.0269+	1.0269+	1.2109+	0.2780+	1.2904+	0.0000+
				0.7081+	

Ultimate moment capacity, kip-ft

	AASHO	BPR
Required	2903.50+	2964.80+
Section capacity	3597.28+	3561.03+

TABLE 10 — DEFLECTION HISTORY

Time	In.	Span/Δ
Initial prestress	0.7468+	
Final prestress	1.2453+	
DL slab	0.4500−	
Final position	0.8888−	
LL	0.3149−	2324.+

gated for its ultimate moment capacity. The design load moments at midspan are multiplied by ultimate load factors as specified by AASHO and BPR. These produce a midspan moment listed under "Required" in Table 9. The section when investigated under procedure recommended by AASHO shows a section capacity as listed. If this section capacity were not larger than the required AASHO value, the fact would be indicated by a flag. The ultimate capacity is then computed substituting BPR criteria and again the comparison of values made.

Deflection patterns—Deflections of the post-tensioned beam at important time intervals are shown in Table 10. For this phase of the beam study the concrete modulus of elasticity was assumed to be

$$E = 1800 + 0.500 f_c'$$

TABLE 11 — WEB DESIGN FOR PRETENSIONED SYSTEM

		Support	Quarter span	Midspan
Diagonal tension study				
Loading				
Prestress	V	0.000+	0.000+	
	M	592.986−	592.986−	592.986−
DL				
	V	33.887+	20.775+	
	M	212.171+	475.238+	633.650+
SDL				
	V	9.029+	5.535+	
	M	56.534+	126.630+	168.840+
LL + I				
	V	45.115+	36.121+	21.111+
	M	252.110+	540.616+	628.455+
Stresses				
σ_t without reinforcing		0.0750+		
σ_t with reinforcing		0.2000+		
Level 2 σ_t		0.126−	0.046−	0.004−
	θ	39.52 +	11.90 +	3.46 +
Level 3 σ_t		0.098−	0.059−	0.008−
	θ	16.92 +	14.58 +	7.01 +
Level 4 σ_t		0.078−	0.057−	0.007−
	θ	14.43 +	15.13 +	6.44 +
A_v per spacing, required		0.0634+	0.0000+	0.0000+
Ultimate shear design				
Loading				
Prestress	V'	0.000+	0.000+	
	M'	592.986−	592.986−	592.986−
DL				
	V'	50.831+	31.163+	
	M'	318.256+	712.857+	950.476+
SDL				
	V'	13.544+	8.303+	
	M'	84.801+	189.945+	253.260+
LL + I				
	V'	112.789+	90.302+	52.778+
	M'	630.277+	1351.540+	1571.137+
A_v per spacing, required		0.0963+	0.0705+	0.0287+

As the concrete at prestress application is usually not full strength, two values of E_c are used in camber computations. The creep coefficient (ratio of total strain to elastic strain at first application of stress) is 2.5. This may be severe depending on manufacturing and construction procedures. If a better value is available in the design phase of a beam it may be substituted. Note the beam rises at application of prestress, and continues after prestressing is completed. During construction of the deck slab, the beam drops as indicated by DL of slab. This drop continues after construction is completed until time-dependent straining (creep) is completed, producing a final deflection value.

The values shown in Table 10 are the results of the post-tensioned beam design. The pretensioned method produces slightly higher cambers. This is because of the added effects of prestressing moment near the supports.

Diagonal tension study—As was stated earlier, economy of material dictates a thin web for prestressed concrete members. At the junction of web to top and bottom flanges and at the centroidal axis of the beam section a study is made for the values and directions of the principal tensile stresses at full design loading. This is done for three

TABLE 12 — WEB DESIGN FOR POST-TENSIONED SYSTEM

		Support	Quarter span	Midspan
Diagonal tension study				
Loading				
Prestress	V	38.268—	22.914—	
	M	240.135—	524.598—	672.333—
DL	V	33.887	20.775+	
	M	212.171+	475.238+	633.650+
SDL	V	9.029+	5.535+	
	M	56.534+	126.630+	168.840+
LL + I	V	45.115+	36.121	21.111+
	M	252.110+	540.616+	628.455+
Stresses				
σ_t without reinforcing		0.0750+		
σ_t with reinforcing		0.2000+		
Level 2 σ_t		0.029—	0.019—	0.005—
θ		9.55 +	7.90 +	4.37 +
Level 3 σ_t		0.027—	0.022—	0.011—
θ		9.96 +	10.05 +	9.53 +
Level 4 σ_t		0.024—	0.022—	0.008—
θ		9.71 +	11.03 +	7.94 +
A_s per spacing, required		0.0000+	0.0000+	0.0000+
Ultimate shear design				
Loading				
Prestress	V'	38.268—	22.914—	
	M'	240.135—	524.598—	672.333—
DL	V'	50.831+	31.163+	
	M'	318.256+	712.857+	950.476+
SDL	V'	13.544+	8.303+	
	M'	84.801+	189.945+	253.260+
LL + I	V'	112.789+	90.302+	52.778+
	M'	630.277+	1351.540+	1571.137+
A_s per spacing, required		0.0755+	0.0581+	0.0287+

TABLE 13 — COMPOSITE SHEAR CONNECTION

	Support	Quarter span	Midspan
v', capacity	0.150+		
V', SDL	16.607+	8.303+	
V', LL + I	125.738+	90.302+	52.778+
v'	0.179+	0.124+	0.066+
Shear key ratio	0.2998+	0.000+	0.000+

locations on the span: at a point near the support, at the quarter point, and at midspan. Tables 11 and 12 show a complete web design for the pretensioning and post-tensioning system. The shears and moments due to each of the design loads are specified. The loadings are divided into groups that effect first the beam (Prestress, DL) and then the composite beam (SDL, LL + I).

Values of the principal tensile stresses are significantly different for the two prestressing systems investigated. The reduction of these stresses near the support in Table 12 is due to the vertical component of the draped-cable used in the post-tensioned system. The allowable stress used in this problem is $0.015 f_c'$ with no reinforcing. If web reinforcing were used the allowable diagonal tension is $0.04 f_c'$. These limits can be changed if necessary. When the diagonal tension stress exceeds the lower limit a steel requirement is indicated. If the stress exceeded the upper limit the value would be flagged and computations continued.

Ultimate shear design—Tables 11 and 12 show the ultimate shear design for both prestressing systems discussed. The shears and moments at ultimate load are shown in the printout. While the moments are not considered in most ultimate shear formulas the fact is recognized that they do affect the shear capacity.

Because the diagonal cracking in the beam produces indeterminate effects on the prestress, the assumption is the prestressing force does not change, from design load to ultimate load. The dead load values are increased by 1.5 and live load values are increased by 2.5 from design load values. Below the loading tabulations are the areas of web steel required according to computations. The final value of web steel area requirements has lower limits which are set by AASHO. In all cases, these limits are observed.

Composite shear connection—Composite shear connection results are shown in Table 13. Not much information is available on design of the horizontal shear connection between the precast concrete and cast-in-place concrete. AASHO specifications require design for ultimate load using the formula

$$v = \frac{V_u Q}{I}$$

with provisions for shear keys if the bond capacity is exceeded.

This is one aspect of prestressed concrete design where attention must be paid to new details. Web steel should continue into the cast-in-place deck to prevent uplift. This steel will also act as a dowel to resist horizontal shear.

CONCLUSIONS

From the results of this program the engineer can extract data sufficient for design. In addition, study of the output data leads to an understanding of stress conditions for various loadings. While accumulating this data is by no means an impossible task by manual computations, it does consist of trial and error procedures, repetitive type calculations, numerical evaluations, and comparisons as well as many straightforward computations. In programing this problem the computations were organized according to a logical design procedure. The result is a design program which has been used successfully for numerous beam designs. Although the program was actually written for the I-beam composite-type structure, it has been modified to suit a box beam type of structure.

The computer time required for a complete program run is from ½ to ¾ hr. Selection of the most economical beam on the basis of a full investigation is therefore the greatest benefit derived from this program.

This program does not consider a new method of pretensioning where the line of the strands is not horizontal but bent around inserts in the forms. The benefits of a post-tensioned draped cable system are then realized in a pretensioned system. However, should it be necessary, the program can be easily adapted to handle this innovation.

Possible extensions for a more complete program are the design of mild steel reinforcing and analysis of stress conditions in the end block.

If further information on the details of the design specifications or programing techniques are desired, inquiries should be addressed to:

Hamden Testing Services, Inc.
610 Bloomfield Avenue
Bloomfield, N. J.

ACKNOWLEDGMENTS

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Title No. 57-65

Response of Concrete Shear Keys to Dynamic Loading

By ROBERT J. HANSEN, EDWARD G. NAWY, and JAYANT M. SHAH

A series of comparative static and dynamic tests on three types of concrete shear keys has been completed. Results indicate that plain concrete shear keys can withstand stresses as high as 2000 psi, that imposing transverse compression on the keys raises the ratio of shear to compressive strength by about 50 percent in the dynamic loading case though by only about 5 percent in the static loading case, and that keys indicate much higher shear strengths when loaded dynamically than statically.

■ SHEAR KEYS ARE REQUIRED in many structures in which separating joints are used. The stresses transmitted through such joints may be of high magnitude and at rapid strain rates in cases in which a structure is required to resist the blast from an atomic weapon.

Tests[1,2,3] have indicated that concrete can offer a high resistance to pure shear stresses. However, no tests are apparently available on shear strengths of concrete keys under dynamic loading. Accordingly a program to investigate this phenomenon was conceived by Holmes and Narver, Inc., acting under a general contract with the Atomic Energy Commission. Massachusetts Institute of Technology was selected to carry out this program under a contract with the AEC utilizing a large capacity dynamic loading machine which was originally designed and constructed at MIT under contract to the Office of the Chief of Engineers.

RESEARCH PROGRAM

Objectives

The basic objectives of this research program were to determine the magnitude of ultimate shear strength of concrete shear keys under static and the dynamic loading comparable to that experienced in a building subject to air blast loading from a nuclear weapon. Three types of keys were considered:

1. Plain concrete
2. Plain concrete under directly imposed transverse compressive stresses
3. Concrete reinforced by diagonally embedded dowels

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Scope

The program was limited to the testing of 18 specimens, six of each of the three types and the correlation and interpretation of this data.

For each of the three types of specimens, two were tested statically and four dynamically.

TEST SPECIMENS

Description of specimens

Fig. 1 shows the details of the specimens used, each of which consisted of a test block cast in a housing frame. Shear keys provided the only connection of the test block with the housing frame, projecting from the test block into the surrounding housing frame. Elsewhere, the test block was separated from the housing frame by a cardboard. The keys were provided with a bearing area equal to the shear area so that failure would be in shear. Reinforcement was used in the test block only for the purpose of preventing shrinkage cracks and was distributed as shown in Fig. 2 and 3.

There were three types of specimens in the test program:

I. Specimens with plain shear keys.

II. Specimens with shear keys under transverse compression. In this type, the housing frame was cast in two parts separated by a thin, well

TABLE 1 — GENERAL DATA ON TEST SPECIMENS

Specimen No.	Category of the specimen	Total area of shear keys, sq in.	Cylinder strength of test block, psi	Cylinder strength of housing frame, psi	Thickness of cardboard used, in.
1	I	72.40	4600	5700	1/8
2	I	72.40	4560	4880	1/8
3	I	72.18	4210	4740	1/8
4	I	72.03	4310	5530	1/8
5	I	71.95	4170	4810	1/8
6	I	72.15	5550	4700	1/32
7	II	72.54	5830	5950	1/8
8	II	73.03	5130	4380	1/8
9	II	72.69	4030	4490	1/32
10	II	73.10	3890	4210	1/32
11	II	73.36	3890	5560	1/32
12	II	73.57	3910	5590	1/8
13	III	73.78	4170	5410	1/8
14	III	73.99	5200	3960	1/8
15	III	73.99	3930	3750	1/8
16	III	74.00	4380	4240	1/8
17	III	74.31	4280	4530	1/32
18	III	74.11	4210	5270	1/32

oiled steel plate. This enabled the transverse force to be transmitted onto the test section. The specimen was compressed between two steel plates which were connected by four aluminum rods as shown in Fig. 1. These tie rods were stressed by tightening the nuts at their ends imposing thus transverse compressive stresses on shear keys. In one half of the specimens 300 psi transverse compression was applied, while in the other specimens only 150 psi transverse compression was applied. These values were chosen to simulate the effect of downward roof loads on sections above the keys.

III. Specimens with shear keys reinforced by diagonal bars. Four #3 deformed bars were employed as shown in Fig. 1, 2, and 3 to aid the keys in taking the load.

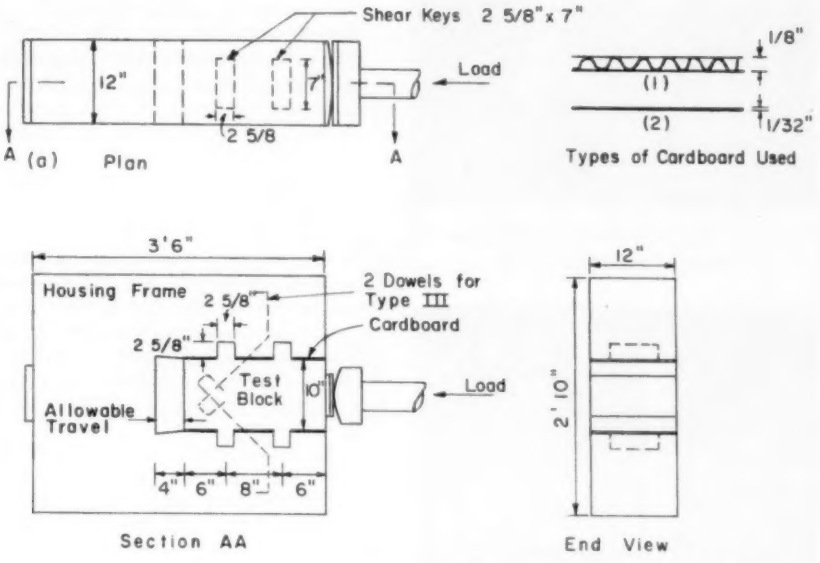


Fig. 1—Details of specimens

Fig. 2 — Reinforcement distribution in test block (Type III)

Fig. 3 — Reinforcement distribution in the housing frame (Bars projecting from the test block are dowels)

Six specimens were cast of each type; specimens of each group were identical except for the compressive strengths of concrete which are given in Table 1.

Properties of materials

Initially, a concrete mix for the test block was designed for a strength of 3500 psi at 7 days. The proportion was as follows:

1 part by weight of high-early-strength portland cement
2.50 parts by weight of sand
3.40 parts by weight of gravel
6.45 gal. of water per sack of cement

The mix used for the housing frame was made stronger and its proportion was as follows:

1 part by weight of high-early-strength portland cement
2.25 parts by weight of sand
3.05 parts by weight of gravel
5.70 gal. of water per sack of cement.

The sand used had a fineness modulus varying between 2.70 and 3.00 and its saturated-surface-dry specific gravity was 2.67. Gravel size ranged between ½ and ¾ in. (about 2 percent of 1-in. size) and its saturated-surface-dry specific gravity was found to be 2.71.

Later on, a study was made of the effect of compressive strength on the behavior of shear keys; hence the water-cement ratio was varied to obtain different strengths. Sometimes, specimens could not be tested at 7 days and in such cases, the compressive strength went above the designed values.

Only a nominal amount of steel was used in test block as well as housing frame. All reinforcing steel used was intermediate grade deformed #3 bars. The same steel was used for the dowels. The yield load of #3 bars was found to be 5300 lb or 48,200 psi.

Preparation of specimen

All the specimens were cast horizontally. The casting of the specimen was done in two parts. First, the test block was cast. After 24 hr, the three side form plates were removed and the keys were then thoroughly cleaned and brushed by a steel brush and then measured. Cardboard, ⅛ in. thick ($\frac{1}{32}$ in. in some specimens), was placed on all sides of the test block except for the shear keys. Then the housing frame was cast.

The specimen and control cylinders were stored under moist conditions for 2 days after which they were removed from the forms and left under normal laboratory air conditions until being tested.

TEST EQUIPMENT

General

For testing the specimens, the large capacity dynamic testing machine, which was designed, constructed and used at MIT under contract with the Office of the Chief of Engineers, Department of the Army, was readily adapted to the test program. The machine consists of an oil hydraulic system capable of producing a single concentrated load up to 300,000 lb with control over the force-time function. Static loading can be obtained by gradually increasing the oil pressure. In a dynamic test, the load pulse is governed by a function generator capable of generating a wide range of load-time functions.

The load pulse employed for this test program was roughly of triangular shape with a slightly flat peak as shown in Fig. 4. The peak of the load was reached in 25 to 40 millisec and the total duration of the pulse was about 2250 to 2750 millisec.

The specimen was supported in the loading machine by various restraints as shown in Fig. 5. It rested on a U-frame while the load on it was transmitted to the frame of the loading machine through a strut. The specimen was also supported laterally by a special attachment which permitted movement in the direction of load only.

Load measurements

Records of the applied load, displacement of the test block relative to the housing frame, load in tie rods in Type II specimens, and load in dowels in Type III specimens, all as functions of time were essential for the study of

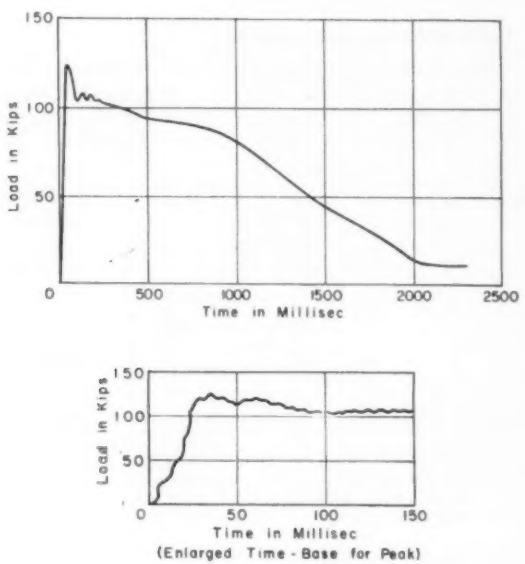


Fig. 4 — A typical load-pulse (third pulse on Specimen 16)

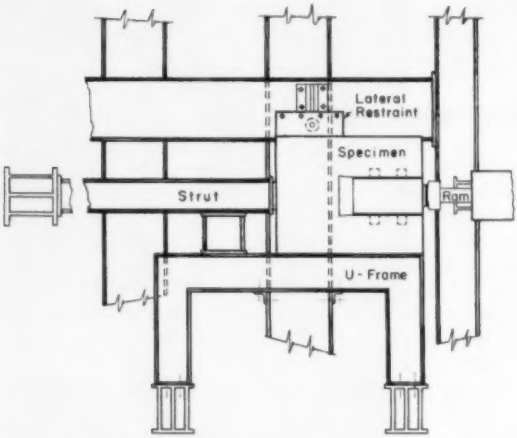


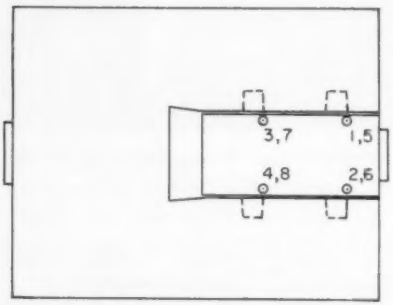
Fig. 5 — Support arrangement for the specimen

the behavior of the specimens tested. As the nature of the tests was dynamic and the magnitude of shear deflections small, a high speed sensitive system had to be devised.

The load was measured by a bridge of eight C-7 strain gages fastened on the loading ram. The signal from these strain gages was fed into dual beam cathode-ray oscilloscopes and permanent records of traces from the screen of the oscilloscopes were obtained from oscillograph record cameras. Two oscillographs were obtained for each load pulse: one with a slow sweep of 1 sec per in. for the purpose of tracing the whole pulse and another on another scope with a faster sweep of about 100 millisec per in. for tracing the initial curve from which the rise time of the pulse and failure time could be determined with greater accuracy.

Deflection measurements

The movement of the test block relative to the housing frame was measured at eight points (see Fig. 6). Electric-inductance gages of the moving-core solenoid type (known as a linear variable differential transformer) were used.



1,2,3,4 are gage-points on the front face while 5,6,7,8 are situated respectively behind 1,2,3,4 on the rear face.

Fig. 6—Arrangement of gages

The output of the differential transformer due to the movement of its core with the test block was amplified. This amplified signal was then fed into the galvanometers of an oscillograph and a continuous trace was obtained.

Stress in tie rods and dowels

Tie rods were of aluminum on which two SR-4 gages were fastened. The readings were taken on a strain indicator at the end of each load pulse during dynamic testing. In the static tests, the readings were taken after each increment of load.

TESTS

General

Preparation of the specimens for testing was the same whether for dynamic loading with rise times to peak load of 25 to 40 millisec or for the static case of 10 to 15 min total load duration. The load in the case of static tests was continuously increased in steps up to failure, while in dynamic tests it was applied in several triangular pulses of constant duration, increasing in magnitude until a peak pulse corresponding to failure was reached. The first blow was a small load to check the equipment. As the dynamic failure was always sudden, a few smaller blows were first applied before causing failure to obtain information about the stress-strain characteristics and general behavior of the keys. This also avoided overhitting the specimen.

STATIC TEST RESULTS

Results of all static tests are given in Table 2, tabulated in the order of the specimen strengths. Typical deflection patterns are shown in Fig. 7, 8, 9, and 10. The shear strength is found to be consistently high and the failure to be brittle.

In Category I, the reason for the higher strength of Specimen 2 compared to that of Specimen 1 appears to be the presence of stronger aggregate at the plane of shearing in the former specimen. The failure plane was as much inclined to the axis of the specimen as permitted by the gap between the housing frame and the test block (equal to the thickness of the separating cardboard).

In Category II, the value of the transverse compression tabulated is the initial value only. During testing as the arms of the housing frame tried to separate, the tie rods were stretched further (yielding

TABLE 2 — STATIC TESTS

Category	Specimen No.	Cardboard thickness, in.	Test block f_c', psi	Housing frame, psi	Load, kips	Total stress in dowels, lb	Transverse† compression, psi	Maximum deflection, in. × 10^5		Shear strength τ, psi	$\frac{\tau}{f_c'}$
								Front row of keys	Rear row of keys		
I	1	1/8	4600	5700	73.6*	—	—	270	180	1020	0.22
	2	1/8	4560	4880	97.5*	—	—	452	471	1350	0.29
II	10	1/32	3890	4210	77 81*	—	150	1086	762	1100	0.28
	7	1/8	5830	5950	110 114*	—	300	365 666	1384 1708	1570	0.27
III	13	1/8	4170	5410	88 92*	18,780	180	729 1274	541 1108	1070‡	0.26
	17	1/32	4280	4530	99 103*	10,630	101	1645 1708	1545 1456	1290‡	0.30

*Failure value

†Transverse compression given for Category II is the value imposed by the tie rods at the beginning of the test while that given for Category III is the value induced by dowels at the time of failure.

‡Shear strength in Category III is calculated after subtracting the horizontal component of the stress in the dowels from the total failure load.

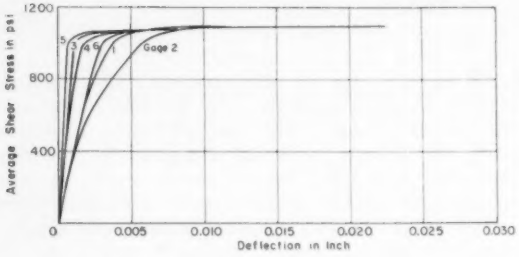


Fig. 7—Plot of shear stress versus deflection, Specimen 10

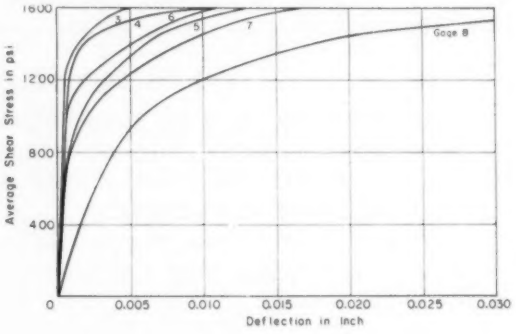


Fig. 8—Plot of shear stress versus deflection, Specimen 7

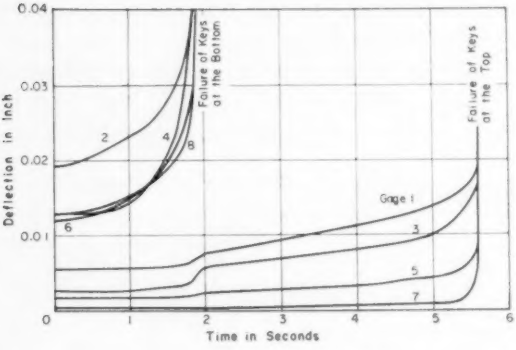


Fig. 9 — Deflection versus time curves at failure, Specimen 10, load at failure was 81 kips

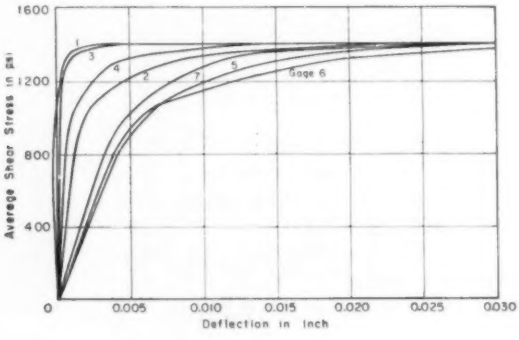


Fig. 10—Plot of shear stress versus deflection, Specimen 17

in Specimen 7), increasing the transverse compression on the test sections, especially at the time of failure. The inclination of the failure plane was not the maximum possible and some crushed powder was found at the test sections. In Specimen 10, uneven distribution of the load among the top and the bottom shear keys was indicated by the fact that the top row of keys failed some time after the bottom keys had failed (see Fig. 9). Also, the bottom keys showed a larger proportion of weak aggregate. In Specimen 7, the gravel at the failure plane was also somewhat weaker. The tendency for pure-shear type failure was more pronounced in this category than in both Categories I and III.

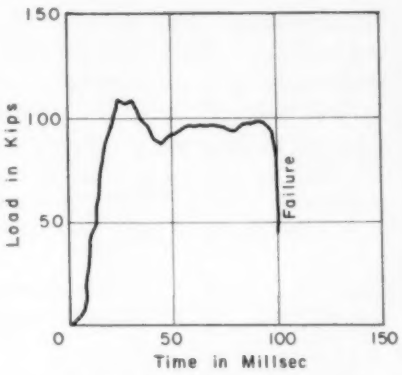


Fig. 11—Plot of load versus time for failure pulse on Specimen 4

In Specimens 13 and 17 of Category III, the shear sections were again under some transverse compression which was induced by the transverse component of the stress in the dowels. The failure plane tended to be inclined to the axis of the specimen. Specimen 17 appeared to have had better aggregate at the failure plane. In both cases, the dowels were below yield point at the time of failure of the shear keys.

The load distribution between the front and rear keys would also be affected by the amount of strain in the housing frame. Too rigid a frame would force the major portion of the load on the front keys resulting in a low average strength. The effect of this factor seems to have contributed to the higher strengths of Specimens 2 and 17.

DYNAMIC TESTS

Results of the dynamic tests are given in Tables 3, 4, and 5. Typical deflection curves are given in Fig. 12, 13, and 14. The manner of failure is indicated by the load-time curves in Fig. 11 and 14.

Dynamic strengths, in general, were greater than the static strengths, especially in Categories II and III; but the influencing factors and the brittleness of failure appear to be similar to that of the static case. Among the specimens of Category I, Specimen 6 had weaker pieces of coarse aggregate at the failure plane. Specimens 3 and 4 had better gravel and in spite of the greater number of dynamic loads, they behaved quite well. Specimens 4 and 6 showed large residual deflections and a considerable time lag between the peak load and the peak deflection. The inclination of the failure plane was maximum in all of these specimens.

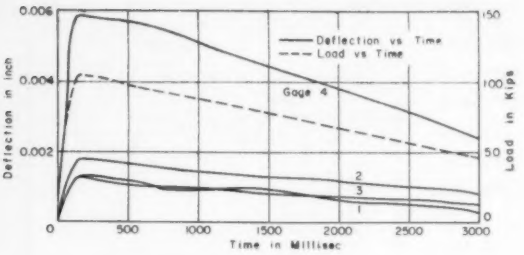


Fig. 12 — Load pulse 6, Specimen 8

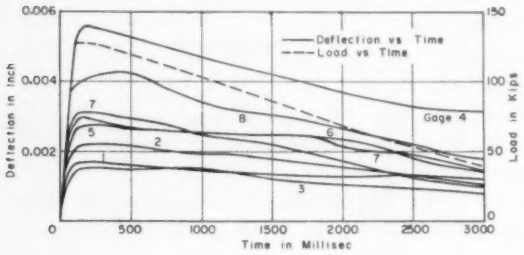


Fig. 13 — Load pulse 8, Specimen 8

In Category II, the specimens showed a striking increase in strength due to the transverse compression. All the tie rods yielded in case of Specimen 8 while half of them yielded in Specimen 9. Again, weak gravel appears to have lowered the strength of Specimen 12. The two blows preceding the last one were of about the same magnitude, and at the fourth blow in this specimen there were large residual deflections recorded indicating extensive damage. There was also a time lag between the peak load and the peak deflection. In the same way, a larger number of blows and a weaker gravel seem to have prevented Specimen 8 from developing a higher strength. Specimens 8 and 9 also

TABLE 3 — DYNAMIC TESTS
CATEGORY I: PLAIN KEYS

Specimen No.	Cardboard thickness, in.	Test block, f_c', psi	Housing frame, f_c', psi	No. of blows	Rise time and failure time, millisec	Load, kips	Maximum deflection, in. × 10^5		Shear strength, τ, psi	$\frac{\tau}{f_c'}$
							Front row of keys	Rear row of keys		
6	1/32	5550	4700	4 5*	40 40*	73 84*	195 350	225 437	1160	0.21
5	1/8	4170	4810	3 4*	25 30*	52 93*	148 360	61 305	1290	0.31
3	1/8	4210	4740	6 12*	50 25*	52 104*	80 447	56 337	1440	0.34
4	1/8	4310	5530	7 8*	25 25 100*†	106 109*	640 1108	569 730	1510	0.35

*Failure value
†When the specimen did not fail at the end of rise-time, but failed some time later, the failure time is given separately. Otherwise, the failure time is the same as the rise time.

showed time lags (smaller in Specimen 9), but only small residual deflections. In all of the specimens, a quantity of crushed powder was found at the test sections. Again, the tendency to pure shear type failure was more pronounced in this category than in the other two.

The specimens of Category III too, showed an increase in strength over those of Category I. Specimen 18 took a long time to fail (300 millisecs). This probably contributed to the reduction of its strength. The strength of Specimen 14 seems to have been reduced due to a

TABLE 4 — DYNAMIC TESTS
CATEGORY II: TRANSVERSE COMPRESSION

Speci-men No.	Card-board thick-ness, in.	Test block, f_c', psi	Hous-ing frame, f_c', psi	No. of blows	Rise time and fail-ure time, milli-sec	Load, kips	Trans-verse† com-pres-sion on keys, psi	Maximum deflection, in. × 10^5		Shear strength, τ, psi	$\frac{\tau}{f_c'}$
								Front row of keys	Rear row of keys		
12	1/8	3960	5590	3 5*	40 50 200*‡	107 110*	150†	1136 1762	1097 1580	1500	0.38
11	1/32	3890	5560	5 6*	40 30*	136.5 137*	150	700 1711	1197 1394	1870	0.48
9	1/32	4030	4490	1 3*	90 30*	65 151*	300	1213	950	2070	0.52
8	1/8	5130	4380	6 9*	110 70*	128 153*	300	240 901	346 1370	2090	0.41

*Failure value
†Transverse compression given is the value imposed by the tie rods at the beginning of the test.
‡When the specimen did not fail at the end of rise time, but failed some time after it, the failure time is given separately. Otherwise, the failure time is the same as the rise time.

TABLE 5 — DYNAMIC TESTS
CATEGORY III: WITH DOWELS

Speci-men No.	Card-board thick-ness, in.	Test block, f_c', psi	Hous-ing frame, f_c', psi	No. of blows	Rise time and fail-ure time, milli-sec	Load, kips	Total stress in dowels, lb	Trans-verse† com-pres-sion in-duced by dowel action, psi	Maximum deflection, in. × 10^5		Shear‡ strength, τ, psi	$\frac{\tau}{f_c'}$
									Front row of keys	Rear row of keys		
18	1/32	4210	5270	2 3*	40 40 300*§	85 121 102*	13,780	132	508 1856	345 1345	1500	0.36
15	1/8	3930	3750	3 4*	60 25*	93 123*	10,600	101	778 1639	810 1865	1560	0.40
14	1/8	5200	3960	8 9*	40 60 75*	120 127*	11,670	112	764 737	757 869	1610	0.31
16	1/8	4380	4240	3 4*	35 35*	125 134*	21,200	202	1135 1415	1074 1335	1610	0.37

*Failure time
†Transverse compression due to the transverse component of the stress in the dowels at failure
‡Shear strength calculated after subtracting the horizontal component of the stress in the dowels from failure load
§When the specimen did not fail at the end of rise time, but failed some time after it, the failure time is given separately. Otherwise, the failure time is the same as the rise time.

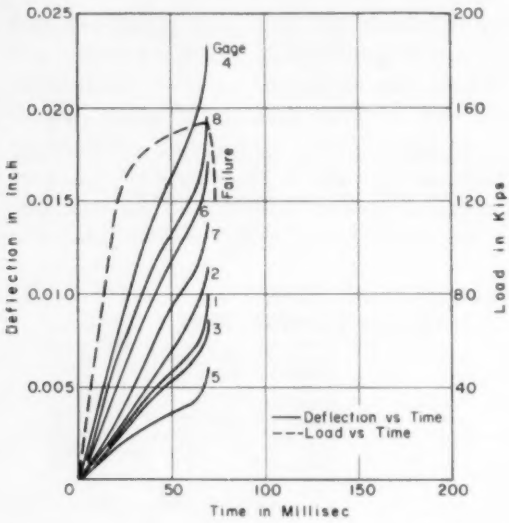


Fig. 14 — Load pulse 9, Specimen 8

larger number of blows. In Specimen 16, unlike others, the dowels were stressed to the yield point at the time of failure. All specimens except Specimen 18 showed a large time lag between peak load and peak deflection. The residual deflections were high in Specimens 14 and 15. The failure plane was inclined.

DISCUSSION OF TEST RESULTS

In studying the behavior of each specimen and comparing and grouping the results recorded, it is seen that an important feature is common to all of them, namely, a high strength of concrete in pure shear, particularly under dynamic loading. The behavior of the plain concrete shear keys appears to be affected by many parameters, viz., nature of the load, its frequency and duration, transverse stresses on shear sections, strength and properties of the aggregate, and amount of freedom for the diagonal tension failure to occur. One or more of these factors dominated the behavior of the specimens tested.

The shear keys exhibited larger strength under dynamic loading than under static loading. The ratio of the shear strength to the compressive strength increased considerably under dynamic loading in each of the three categories of the specimens as seen in Table 6. This increase was more than 50 percent above the static case in the second category where the transverse compression was applied through the tie rods. The specimens with the dowels registered considerably less increase (about 28 percent) while the increase was the smallest (only 15 percent) in the case of plain shear keys. The tendency for the

TABLE 6 — RATIOS OF SHEAR TO COMPRESSIVE STRENGTH

No.	Category	Static	Dynamic	Percent Increase
I	Plain	0.26	0.30	15
II	With transverse compression			
	150 psi	0.28*	0.43	54
	300 psi	0.27*	0.46	70
III	With dowels	0.28	0.36	28

*Values based on the result of only one test

TABLE 7 — EFFECT OF TRANSVERSE COMPRESSION OR DOWELING ON INCREASING SHEAR TO COMPRESSIVE STRENGTH RATIOS

Loading	Transverse compression type over plain type, percent		Doweled type over plain type, percent
	150 psi	300 psi	
Static	7	4*	7
Dynamic	43	53	20

*Values based on the result of only one test.

diagonal tension or the pure shear type failure, and the brittleness of failure appear to be unaffected by the dynamicity of the load.

The transverse compression on the shear keys appears to play an important part in improving their behavior, particularly when the load is dynamic (see Table 7). The specimens with higher transverse compression did give higher shear strengths. The dowels appear to be helping the shear keys in two ways, the horizontal component of the tensile force developed in them directly taking up a part of the load on the specimen and its transverse component inducing transverse compression on the shear keys and thus improving their behavior. The failure of the shear keys under transverse compression is found to be brittle, but to a lesser degree than that of the plain shear keys. The deflection of the plain shear keys near failure ranged between 0.003 to 0.011 in. while that of the keys under transverse compression or with the dowels ranged between 0.009 to 0.018 in. Also the tendency for diagonal tension failure was greatly reduced when transverse compression was present.

The high shear strength of the shear keys supports the experimental results of the static biaxial and triaxial tests on concrete and mortar cylinders. The data from the present investigation is too limited to allow any quantitative correlation with the previous literature. The actual mechanism of failure is a complex phenomenon. It might well be along the lines similar to those explained by Brandzaeg's[2] hypothesis for anisotropic materials. Local failures seem to be preceding the total failure as evidenced by the residual deflections observed. They would start at points of stress concentrations. But the progress of yielding of the material is checked by the surrounding understressed elements which are still in the elastic zone. A redistribution of existing nonuniform stresses would follow within the material transferring more of the load to the understressed elements and thus leading to the propagation of yielding,[2,4] until complete failure is precipitated. The ultimate load is raised due to this internal redistribution of stresses between the gel and the stone. Due to heterogeneity of concrete, the actual stress condition at any point will depend on the pattern of

arrangement of the different ingredients of concrete at that point and local failure might be brought about by shearing, splitting, or crushing. But total failure would be accomplished by shearing off the material in the key along the imposed direction of failure.

The strength of a concrete cylinder in a compression test depends upon its resistance to sliding and splitting as proposed by failure theories.[2,3,6] Sliding at any point will occur either by failure of mortar, gravel, or of bond between gravel and gel, whichever is the weakest. Hence, in case of strong gravel, failure will occur in mortar or bond leaving the high strength of gravel not fully exploited. But, in the specimens tested under the present program, strong gravel has to fail if it happens to be at the imposed path of failure. This is supported by the fact that the shear strength to compressive strength ratio is found to be considerably higher than that found by the former investigators. Table 6 shows that the average value of shear strength is 26 percent of the compressive strength under the static load and 30 percent under the dynamic load, while this was found to be 20 to 23 percent on the previous static tests on cubes and cylinders.[3,5,6] This ratio can be as high as 46 percent under dynamic loading when transverse compression is imposed as shown in Table 6. As it seems that the direct shear strength of concrete is influenced by the arrangement of the different elements within the concrete in a somewhat different way as compared to their influence on the compressive strength, it might not be entirely proper to express the direct shear strength in terms of the compressive strength. But this seems to be a convenient way of describing the relative action of the various factors.

Effect of gravel

It is believed that this ratio of shear to compressive strength might be considerably higher than 46 percent if special selected type of gravel is used. The effect of the gravel appears more significant when transverse compression was imposed, since, in that case, a good shear resistance is offered by the gravel firmly interlocked at the shear sections by the transverse compression. The tie rods yielded in some specimens probably because the crushing of the strong gravel produced a large transverse push on the arms of the housing frame which were restrained from moving apart by the tie rods. In such cases, some quantity of crushed powder was always found at the failure planes of broken keys.

The residual deflection increased with each load pulse as a result of progressive local failures of the material and consequently the failure load decreased as a greater number of the load pulses was applied to the specimen. The failure load was also reduced when the peak load of the dynamic pulse was sustained for a longer duration. It is noticed that the load-time and the deflection-time curves in the dy-

namic tests follow a linear pattern except in the decay-time period in the case when dowels were used.

The rigidity of the housing frame also appears to affect the behavior of the shear keys as the compressive strains in it affect the distribution of load on the keys. When the housing frame is stiffer than the test block, the magnitude of the induced strain within the frame is comparatively less than that in the test block. Thus, the front vertical row of keys which are locked within the frame are prevented from transmitting a large portion of the load to the rear keys since the frame has to move in a manner incompatible with the rest of the test block. When the frame is less stiff than the test block, the front row of keys can move bodily with the frame by the amount equal to the compressive strain in the frame, thereby reducing the shear strain in it and transmitting greater load to the rear keys.

The thickness of the cardboard separating the test block from the housing frame does not appear to be important when a large transverse compression is present; but in the case of a little or no transverse compression, failure tended to approach the diagonal tension failure as much as the cardboard thickness permitted.

The cardboard separated the test block from the housing frame everywhere except at the shear keys. It is believed that this eliminated the variable parameter of friction between the test block and the housing frame. Usually, in actual structures, friction at the joints will also contribute to the shear resistance of the joint.

From the preceding discussion, because of the various parameters influencing the failure of the keys, it appears worthwhile that further tests be conducted in the future to confirm the interpretations.

CONCLUSIONS

The following principal results are presented due to their prevalent uniformity among most of the tests:

1. Plain concrete keys have high strength in shear. This is found to be as high as 29 percent of the compressive strength in the case of static loading, and 46 percent and possibly more in the case of dynamic loading (when compression is imposed). This implies that the keys under certain categories of stressing would be more than 50 percent stronger under dynamic pulses than under static loads.

2. Transverse compression on keys and doweling appear to be helpful in increasing the shear resistance as well as making the failure less brittle. One of the important effect of dowels seems to be their inducing transverse compression on the keys.

3. It appears that the quality and strength of the coarse aggregate rather than the compressive strength of concrete governs the strength of the keys in shear. Bond failure of the cementing gel indicating

stronger gravel gives higher strengths of keys than the shearing of weak gravel.

4. Failure of the keys in shear is of a brittle nature both in static and dynamic tests; the deflections recorded being of comparatively small magnitude ranging between 0.003 and 0.018 in.

5. When the housing frame is appreciably stiffer than the test block, the front vertical row of keys appears to take a larger portion of the applied load and fail prior to the rear row of keys. The opposite appears to be true when the test block is stiffer as compared to the housing frame.

6. The thickness of the cardboard separating the specimen from the housing frame influences the mode of failure primarily when the specimen is not under direct transverse compression. An increase in its thickness increases the tendency for diagonal tension failure while a decrease increases the tendency for pure shear failure.

7. Failure load is reduced with the increase in the number of blows applied to the specimen and with longer peak load duration in any load pulse.

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Title No. 57-66

Concrete Properties Relevant to Reactor Shield Behavior

By C. P. THORNE

Available information on the factors affecting the properties of concrete relevant to the determination of the stresses in a concrete reactor shield is examined in detail, and the need for further investigation of several of these properties is noted. As a result of this examination it is possible to state the type of concrete best suited to the particular requirements of reactor shielding. The effect of partial drying of the shield on the distribution of temperature and unrestrained strain is examined theoretically and it is shown that the distribution of moisture content in the shield is of major importance.

■ WITH THE ADVENT OF THE commercial use of nuclear reactors, it is becoming increasingly important to make economical use of the various materials employed. Better knowledge of the behavior of concrete reactor shields would allow increased use to be made of their structural properties, and allow them to carry heat loads considerably higher than the conservative values designers are at present forced to use. In present reactor designs, approximately 90 percent of the radiant energy escaping from the core and reflector system is absorbed in a thermal shield which up to the present has proved a particularly expensive item. An increase in the permissible heat load on the concrete biological shield could allow the size, and hence the cost, of this thermal shield to be considerably reduced.

In this paper, the concrete properties relevant to each of the stages of thermal stress determination, namely, temperature distribution, and unrestrained and restrained strains, are listed and a review is given of the present state of knowledge of the various factors influencing these properties. In addition, the effect of drying of the concrete on the temperature distribution and unrestrained strains in a shield are quantitatively examined.

Damage to concrete arising from its exposure to radiation and high temperatures is not discussed at length since the several test results reported for radiation damage to concrete indicate that such damage is unlikely to be serious at the temperatures normally occurring in a reactor shield (up to 200 C).

HEAT RELEASE AND TEMPERATURE DISTRIBUTION

The core and reflector system of a nuclear reactor emit large quantities of high energy radiation, mainly in the form of neutrons and gamma rays, almost all of which must be absorbed by shielding. The energy of the radiation absorbed by the biological shield is liberated in the shield as heat, and since the energy absorption varies exponentially with distance, the greater part of this heat is liberated in the first tenth of the shield thickness. To protect the biological shield from overstressing or excessive heating, a thermal shield, usually consisting of two steel plates between which cooling air is passed, is interposed between the core and the biological shield. Iron is usually chosen for the thermal shield material because it is a good absorber of high energy gamma rays and has a high cross section for the inelastic scattering of fast neutrons, and these two radiations account for the major portion of the energy escaping from the core. In present design about 90 percent of the energy escaping from the core[8,10] is absorbed in the thermal shield. But improvements in the knowledge of the behavior of concrete in a shield and in design methods could probably reduce this figure considerably.

The properties of concrete relevant to the determination of temperature distributions are:

(a) The rate of neutron attenuation

(b) The relaxation length for gamma rays (i.e., the thickness of concrete required to reduce the intensity by a factor of $e = 2.718$)

(c) The conductivity, for the determination of steady state conditions

(d) The diffusivity, for the determination of transient conditions

Neutron attenuation

The neutron attenuation characteristics of a concrete depend on the neutron energy spectrum, and on the chemical nature of its constituents. The most important item is, however, the hydrogen content, since hydrogen is by far the best moderator. The hydrogen content of concrete is usually in the form of moisture and as a result the dry-

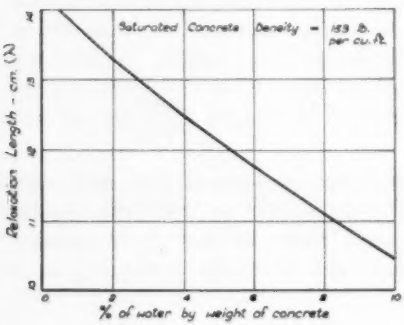

Fig. 1a—Relationship, fast neutron relaxation length (computed from effective neutron removal cross sections) and water content (see Reference 1)

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ing out of the concrete has a severe effect on the relaxation length of fast neutrons. For example, a decrease in moisture content of 5 percent can cause an increase of 30 percent in the relaxation length as shown in Fig. 1.

Gamma ray attenuation

The relaxation length for the attenuation in concrete of gamma radiation is not nearly so dependent on chemical composition as it is for neutrons; for gamma radiation of about 3 Mev* the relaxation length varies almost directly with the inverse of the density.[1] Thus a change in density as a result of moisture loss would cause an increase in the gamma relaxation length of only the same order as the change in density.

Conductivity

The available data suggest that the conductivity of a concrete is determined by the conductivity of the aggregate, the mix proportions, the type of cement, the moisture content, the degree of air entrainment, and possibly the temperature of the concrete. However, little information exists in the literature on the detailed effect of any of these variables on the conductivity of concrete. It is stated in several references that, as an approximation, conductivity increases linearly with density; though as shown in Fig. 2, this is not the case for concretes made from aggregates of varying composition, although it may hold for concretes with aggregates of substantially the same composition, but with varying porosity. The results of tests made in the Civil

*1 Mev (million-electron-volt) is the amount of energy which would be acquired by an electron in falling through a potential of 1,000,000 v. It is equal to 1.6×10^{-6} watt-sec.

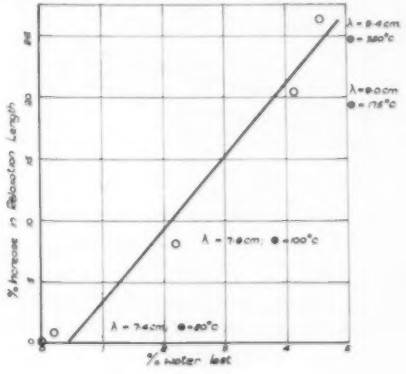

Fig. 1b—Experimental values for fast neutrons; iron-limonite concrete (see Reference 1). Saturated concrete density = 264 lb per cu ft. Measurements made at temperatures indicated, with corresponding equilibrium moisture contents

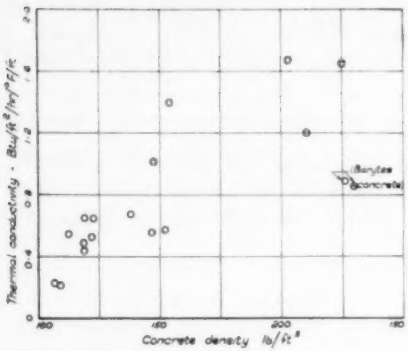

Fig. 2—Conductivity of concrete and density of concrete (data from several sources)

Engineering Department, University of Sydney, on the variation of thermal conductivity and moisture content of concrete made with several different types of aggregate, together with results reported by Tyner[2] for concrete using limerock as aggregate, are given in Fig. 3; they show that a considerable drop in the thermal conductivity, 40 percent in the case of dolerite concrete, is caused by a drop in moisture content from 20 percent by volume to zero. They also support the theoretical prediction that the higher the conductivity of a concrete, the higher will be the drop in conductivity corresponding to the loss of a given percentage by volume of water.

Diffusivity

The diffusivity of a concrete is its conductivity divided by the product of its specific heat and density. The factors determining the conductivity have already been discussed; the density depends on mix proportions, specific gravity of aggregates and moisture content. The specific heat of a concrete may be taken as equal to, Σ(Specific heat × Quantity of constituent per pound of concrete). Concretes with high water contents will, on this basis, have high specific heats because of the high specific heat of water (see Fig. 4). Little information is to be found in the literature beyond values for the diffusivity for specific mixes.

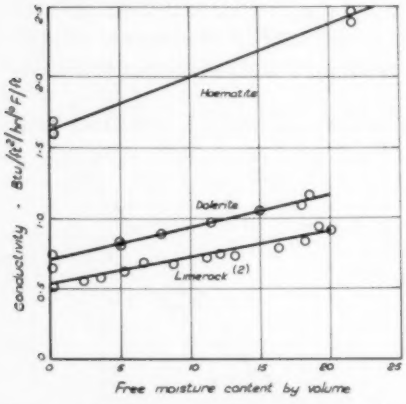

Fig. 3—Change in thermal conductivity due to change in moisture content for concrete containing different types of aggregate

From the above discussion it will be seen that changes in the moisture content of a given concrete will influence all the properties relevant to the temperature distribution in the shield. Experiments at the University of Sydney[21] on the mechanisms by which concrete dries have indicated that if a block of concrete is heated on one face it does not dry uniformly, but dries so that there is a sharply defined

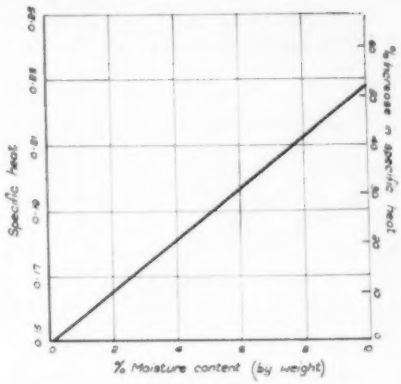

Fig. 4—Theoretical specific heat and moisture content. Above values are computed values; specific heat of dry concrete is assumed to be 0.15, a value often quoted in the literature. Specific heat at moisture content percent m is calculated from: S. H. $= (0.15 + m)/(m + 100)$, where S. H. of water $= 1.0$

dry zone which extends back from the hot face as drying proceeds. To gain an idea of the possible implications of this, temperature rises in a partly dry shield have been calculated assuming that the drying proceeds slowly enough for transient conditions to be ignored, an assumption which is probably justified when the shield has dried past the "hump" in the temperature distribution.[21] Details of these calculations are given in the appendix.

The shape of the temperature rise distribution in the shield does not change greatly as drying proceeds, except that there is a kink in the distribution for a partially dry shield caused by the sudden change in conductivity at the junction of the wet and dry zones (see Fig. 5a). The effect on the maximum and average temperature rises is more

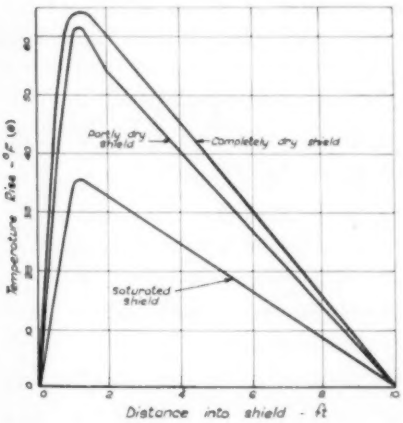

Fig. 5a—Temperature distributions in a saturated shield; a partly dried and a completely dried shield

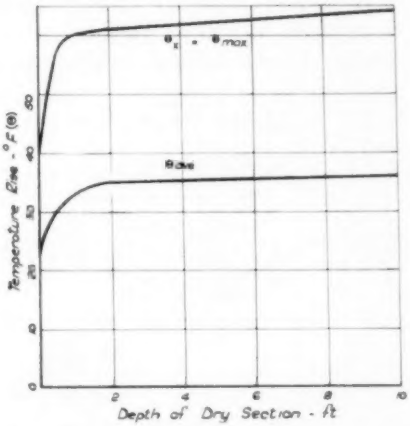

Fig. 5b—Effect of depth of dry section on maximum and average temperature rise in shield

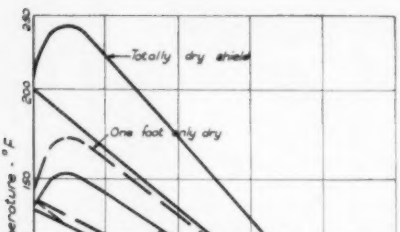

Fig. 6a—Temperature distribution; constant heat flux at inside face

extreme, their values increasing by 80 percent when the first 10 percent of the shield has dried (see Fig. 5b). In addition, the elevated temperature of the core gives rise to a linear temperature distribution to be superimposed on that caused by attenuation. If it is assumed that the inside face is kept at a constant temperature, then the increase in temperature is equal to the increase in temperature rise due to attenuation when the shield is dry and slightly less when it is partially dry (see Fig. 6a). If, instead, the heat dissipated through the shield is constant, then drying the shield has a far greater effect (see Fig. 6b).

UNRESTRAINED STRAINS

Component unrestrained strains

The next step in design is to calculate the unrestrained strains of the concrete in the shield. These unrestrained strains arise from:

(a) Temperature change
(b) Curing
(c) Shrinkage as a result of expulsion of moisture from the gel pores

It appears that in present designs the first of these causes of unrestrained strain is considered to be the major factor, but the others should be considered as contributory factors.

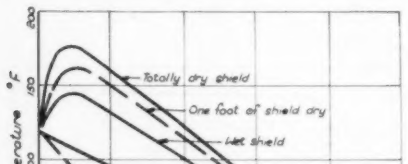

Fig. 6b—Temperature distribution; constant temperature at inside face

Temperature change—The coefficient of expansion appears to depend on the following:

1. Aggregate
2. Cement type
3. Mix richness
4. Degree of saturation
5. Temperature

Of these variables, the first has the greatest effect providing the moisture content is kept constant during the testing period. The values of

coefficient of expansion of different aggregates vary greatly, siliceous materials such as quartz have the highest coefficients while limestones have the lowest. The exact values vary widely with the source of aggregate but typical figures[11] are those shown in Table 1.

TABLE 1 — COEFFICIENTS OF EXPANSION OF AGGREGATES

Aggregate	in. per in. $\times 10^{-6}$ per deg F
Quartzite	6.5 to 6.6
Sandstone	5.6 to 6.5
Granite	3.3 to 5.3
Basalt	4.0
Limestone	2.5 to 3.9

There is no possibility of giving a simple figure for the thermal coefficient of expansion of concrete, unless the aggregate type is first defined. Typical values range from 4.0×10^{-6} in. per in. per deg F for limestone concretes to 8.0×10^{-6} in. per in. per deg F for the quartz concretes.[3,11,12]

The effect of cement type is not well brought out in the literature, since each investigator, with one exception, has confined his studies to one type of cement. An estimate of the importance of this variable can be obtained from a comparison of the results of different investigators[3,11,12,13] for the same type of aggregate. A summary of such results appears in Table 2 for concrete made with siliceous aggregates, and a study of these results shows that the influence of the cement is not great (see Fig. 7a). It has been suggested that a good approximation to the thermal coefficient is the mean of the coefficients of expansion of the aggregate and the cement paste, weighted according to their percentage volume of the solid concrete, and the experimental work reported by Walker, Bloem, and Mullen,[3] supports this concept (see Fig. 7b). It has been found that oven dried specimens and saturated specimens give similar results and therefore the above approximation holds for both cases.

In the preceding discussion it has been assumed that no change in moisture content occurred during the test. The greater part of available information concerns either oven dried specimens or completely saturated specimens; however, from the few results available it appears that concrete with an intermediate moisture content has a coefficient somewhat higher than either of the above extremes.[11]

The effect of temperature on the thermal coefficient is difficult to separate from shrinkage effects, but the little information available[6] indicates that it increases slightly above 300 C.

To sum up, the unrestrained strains of concrete resulting from temperature rise can be predicted with reasonable accuracy, (5 to 10

TABLE 2—VARIABLE OF EXPANSION OF SILICEOUS AGGREGATE CONCRETES

Coefficient	Coefficient of neat cement	Richness of mix
in. per in. per deg F	in. per in. per deg F	
6.2×10^{-6}	5.4×10^{-6}	Variable
5.9×10^{-6}	6.7×10^{-6}	1:6
6.8×10^{-6}	8.2×10^{-6}	1:6
6.9×10^{-6}	10.1×10^{-6}	1:6

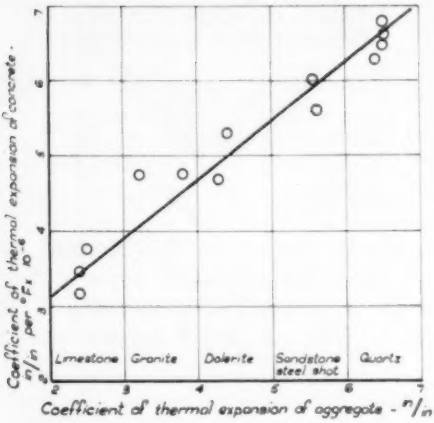
Coefficient of thermal expansion of concrete - in/in per °F x 10⁻⁶
Limestone
Granite
Dolerite
Sandstone steel shot
Quartz
Coefficient of thermal expansion of aggregate - in/in per °F x 10⁻⁶

Fig. 7a—Thermal expansion of concrete (see Reference 11 and 12)

percent) provided the moisture condition of the concrete remains unaltered.

Curing—Concrete cured under saturated conditions undergoes an expansion of the order of 10^{-4} in. per in.[12,17] during 3 years after casting; however, the major portion of the expansion takes place in the first year. Allied with this expansion are the thermal strains associated with the temperature rises caused by the liberation of heat of hydration of the cement. Both of these phenomena have been thoroughly investigated in connection with mass concrete dams and their effects can be evaluated with confidence. It is unlikely that a reactor shield would be required to withstand full operating conditions within a year of casting, and therefore the above problem can be considered separately from that of structural behavior under operating conditions. For these reasons the matter will not be further discussed in this paper.

Shrinkage as a result of expulsion of water from gel pores at elevated temperatures—The subject of shrinkage strains other than setting and curing shrinkage has not been fully investigated. Results reported by Davis[12] and Walley[15] give a figure of the order of 6×10^{-4} for the shrinkage strain caused by drying in air, the zero being taken from a 2-day old specimen. Blakey[4] reports shrinkage strains of the same order for specimens between the ages of 1½ to 28 days, during which time approximately 50 percent of the original water was lost by evaporation. It has been noted that such strains are at least partly reversible,[12] and that no appreciable shrinkage occurs in mortar until the retained water is reduced to some 45 percent of the volume of the cement phase.[4] The above considerations suggest that this shrinkage occurs because of the expulsion of water from the gel capillaries only, as distinct from the capillaries resulting from excess mixing water. Experimental work carried out at the Civil Engineering Department,

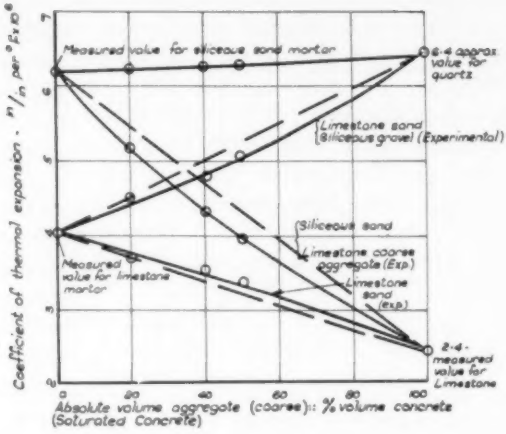
Measured value for siliceous sand mortar
6:4 approx value for quartz
Limestone sand
Siliceous gravel (Experimental)
Siliceous sand
Limestone coarse aggregate (Exp.)
Limestone sand (Exp.)
Measured value for limestone mortar
2:4 measured value for Limestone
Coefficient of thermal expansion - in/in per °F x 10⁶
Absolute volume aggregate (coarse): % volume concrete
(Saturated Concrete)

Fig. 7b — Variation of coefficient of expansion of concrete with coefficient and proportion of aggregate (see Reference 3)

University of Sydney, on the shrinkage of hematite concrete has supported this concept and has shown that as a first approximation, there is a linear relation between shrinkage and moisture content for any given concrete as in Fig. 8.

Resultant unrestrained strains

Assuming that strains induced by curing may be considered separately, two sources of unrestrained strains remain to be considered in the design of the shield. These sources are the expansion due to temperature rise and contraction due to decrease in moisture content.

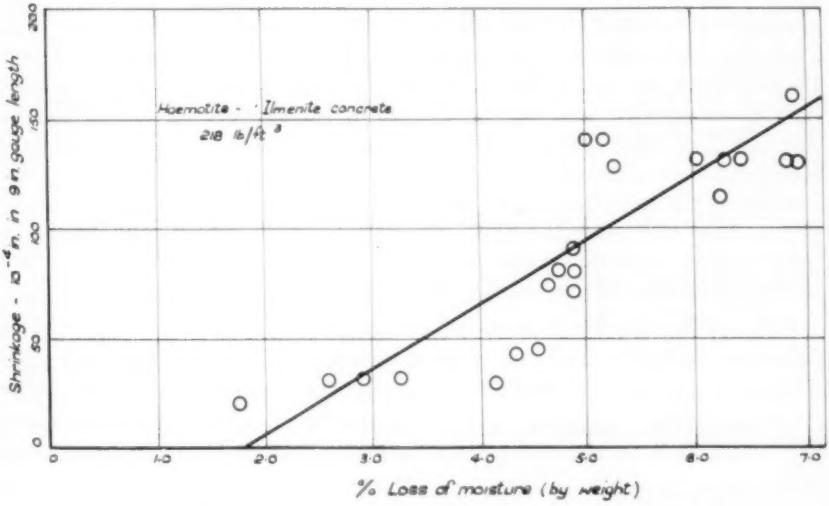

Fig. 8—Relation between shrinkage and moisture content

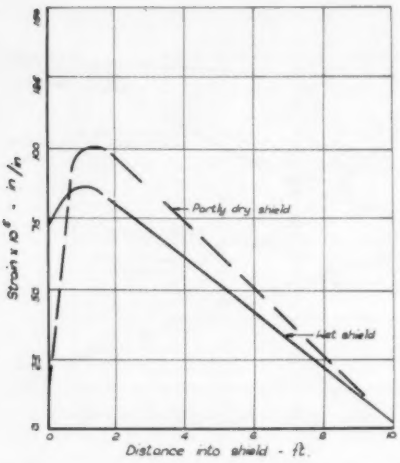

Fig. 9—Unrestrained strains (thermal and shrinkage strains) in a saturated and in a partly dried shield

No definite information has appeared in the literature on the movement of moisture in concrete at elevated temperatures. Raphael[19] has estimated that a biological shield would take a number of years to dry out. It should be noted however, that even partial drying can have an effect on the temperature distribution and on the unrestrained strain distribution, which, considering the size of shrinkage strains could be important. To illustrate the possible magnitude of these effects, unrestrained strains are plotted in Fig. 9 for a completely saturated shield and for one in which the first 10 percent is dry. The calculations are based on a coefficient of thermal expansion of 6×10^{-6} in. per in. per deg F, a shrinkage strain of 6×10^{-4} in. per in. at the inside face of the shield, decreasing linearly to zero at the moisture front, and the temperature distributions previously calculated.

The mechanisms and rate of moisture movement in concrete at temperatures of 100 C and greater have been under investigation at the University of Sydney. The experiments have been carried out on slabs and cylinders of mortar heated in a large gas oven at temperatures ranging up to 180 C. These experiments have yielded data on the variation of temperature and moisture content in the specimens with time, and in all cases the evaporation from the heated face has occurred so rapidly, and has absorbed so much heat as latent heat of vaporization, that temperature equilibrium was not reached until the specimen was completely dried. It is unlikely that such severe interference with heat diffusion would occur in a thick concrete shield; but, if free evaporation is possible at the inside face of the shield the first foot or so of the shield will dry out after a week at temperatures of 300 F. Therefore, shrinkage strains must be considered.

For a shield in which evaporation from the inside face is prevented, moisture diffusion will occur at a much slower rate than will heat diffusion and hence temperature equilibrium will be reached before any appreciable moisture movement has taken place. Hence it would not be possible to take advantage of any relief of thermal strains that might be affected by shrinkage strains. Also, in this case the combination of high temperatures and high moisture contents possible in

a thick shield but impossible in small unsealed laboratory specimens, suggest that steps should be taken to separate temperature and moisture effects on relevant properties, since at present it is usual to conduct tests at elevated temperatures with the specimen at its equilibrium moisture content for that temperature. The main difficulty in carrying out tests on saturated specimens at elevated temperatures is to find a successful method of sealing in the moisture. This is particularly difficult to do because of the high pressures developed by confined water at high temperatures, and it may be necessary to resort to large specimens which can be tested before they dry out.

RESTRAINED STRAINS

Resultant strains

Because of internal and external restraints the final distribution of strain through the shield is different from the unrestrained strain distribution, and it is the difference between these (the restrained strain), that gives rise to stresses in the shield. The magnitude and distribution of these strains depend on the geometry of the shield and the external restraints acting on it.

A rigorous analysis of restrained strains and stresses would include the effect of creep and of the variation of the elastic modulus through the shield: such an analysis would be extremely complex and is outside the scope of this paper. Simplified analyses of stresses in a shield, assuming concrete to be a simple elastic medium with a linear coefficient of expansion, have been done by Davis[1] and Bonsall.[20]

The following discussion is concerned with those concrete properties concerning the values of the restrained strains, viz.,

(a) Elastic modulus
(b) Amount of stress relief afforded by creep

Elastic modulus—The most important factors affecting the modulus of elasticity of a concrete are its mix properties, aggregate type, cement type, water-cement ratio, moisture content at test, temperature at test, and temperature history. It is not proposed to discuss the first five factors here since they have been widely discussed in text books. The last two factors have only recently been studied fully especially at temperatures in excess of 100 C and are therefore worthy of further comment.

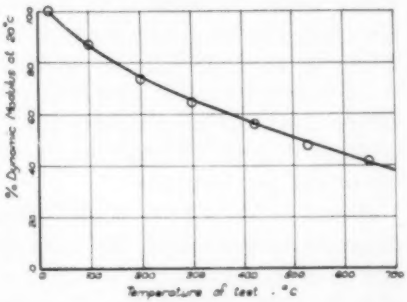

Fig. 10—Effect of temperature on dynamic modulus (see Reference 6)

As regards the temperature at test, the elastic modulus of concrete

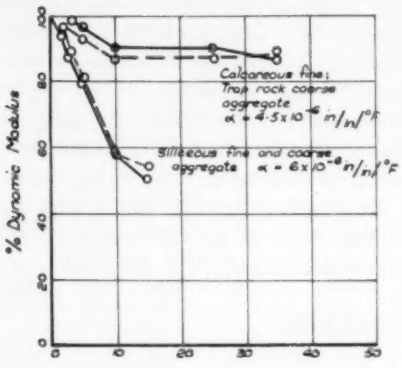

Fig. 11—Cycles from 40 to 140 F in water (see Reference 3)

drops quite sharply as the temperature rises, (see Fig. 10), which shows a typical relation between sonic modulus and temperature.[6] The form of this relationship is independent of the test method.[7] It has been shown that for a given concrete, the modulus at temperatures in excess of 450 C tends to be independent of age or curing conditions. This result was obtained from concrete in which the aggregate used was a calcareous one which decomposed with the evolution of carbon dioxide at 540 C, thus bringing in another variable at high temperatures. The effect of water-cement ratio on the modulus is still appreciable at high temperatures. The reduction is not as great for lean mixes as for rich mixes; the type of aggregate has little effect, providing it does not melt or decompose. It should also be noted that all experiments conducted to date have been carried out at the equilibrium moisture content for the temperature of test.

If concrete is subjected to temperature cycles, its modulus decreases rapidly (see Fig. 11). Low temperature cycles in water from 40 to 140 F can cause a reduction of 40 percent after 10 cycles and the effect of high temperature cycles is even more severe. There is reason to believe that deterioration in these low temperature cycles is caused by thermal stress.[3]

From these data it is evident that a drop in elastic modulus of up to 30 percent may be expected in the high temperature portions of the shield providing they have dried out. The behavior of concrete under conditions of both high moisture content and high temperatures, which is possible in a shield if the inside face is sealed in any way, has not been reported in the available literature presumably because of the difficulty of sealing laboratory specimens.

In a shield subject to cycles of temperature even greater reductions in the modulus may be expected and it may well be that the highest stresses in such a shield would occur when the shield first reaches its operating temperature.

Stress relief—Creep strains—Since the unrestrained strains are time dependent because of the time required for heat and moisture diffusion, the restrained strains and hence the stresses in the shield will also be time dependent. The issue is further complicated by the fact that while the components of the unrestrained strains are independent of the rate of build up of their various causes, the stress producing

strains are dependent on this rate because of the creep characteristics of the concrete.

Even at room temperatures the theory of creep or time-dependent strain of concrete is incomplete, indeed there is no great agreement between the various theories presented in the literature on this subject.[5] When it is considered that in a shield the added complication of varying temperature and moisture content arises, it becomes obvious that a great deal more work is needed before a complete evaluation of creep effects can be made.

TABLE 3 — VARIOUS STRAINS AT FAILURE

Unrestrained shrinkage strain	59.0×10^{-5} in. per in.
Steel strain (resultant strain)	22.0×10^{-5} in. per in.
Restrained strain	37×10^{-5} in. per in.
Elastic strain $= \frac{\text{Concrete stress at failure}}{\text{Experimental value of } E}$	11×10^{-5} in. per in.
Plastic strain	26×10^{-5} in. per in.
Observed concrete stress at failure	380 psi
Restrained strain $\times E$	1480 psi

However, available information does indicate that considerable stress relief is available and to illustrate this an examination of one of Blakey's experiments[4] has been made. The specimen for this experiment consisted of a concrete block with a steel bar cast into it to restrain the shrinkage; a strain gage was attached to the steel bar so that both stress and resultant strain could be measured. Comparison specimens without any restraining bar, were used to determine the sonic modulus of elasticity and unrestrained shrinkage strains so that it was possible to evaluate the elastic strain and the creep strain at every stage up to failure. These experiments, which covered three water-cement ratios, suggested that failure occurred at a constant elastic strain of 11×10^{-5} in. per in., and that the primary failure was in the cement paste.

In Table 3 is given a typical set of the figures for the various strains at failure and these show the considerable stress relief afforded by creep, even though the entire test took only a few days. In all cases it was found that the creep rate was nearly equal to the shrinkage rate as failure was approached, and in a few cases these two rates were equal so that no failure occurred at all. Considering the slow straining rate for a mass structure such as a biological shield, the stress relief afforded by creep could be considerable and some allowance could be made in design for this; however, before it is safe to do this further investigation would be required to obtain the necessary data.

CONCLUSIONS

An attempt has been made to summarize the present state of knowledge as contained in the available literature on the properties of concrete relevant to the structural behavior of a mass concrete shield. It is evident that conductivity, shrinkage resulting from evaporation at elevated temperatures, time-dependent strain in both tension and

TABLE 4 — CONCRETE FOR REACTOR SHIELDS

	Attenuation	Thermal conductivity	Thermal expansion	Drying shrinkage	Heat of hydration	Modulus of elasticity	Creep	Heat and radiation damage	Maximum strain at failure	Homogeneity
	Conditions giving a desirable value for the property listed									
	High density aggregate; good curing to ensure sufficient moisture retention at high temperature	Aggregate with high thermal conductivity; e.g., iron ore; saturated concrete	Aggregate with low thermal expansion	Lean mixes; prepacked placement, low-mixing water	Lean mixes; low heat cement	Lean mixes; high water cement ratios	Rich mixes porous aggregate	Lean mixes; concrete with low thermal expansion; aggregate resistant to heat and radiation	Lean mixes	Prepacked placement, nonsegregating plastic mixes; good control
	Conditions giving an undesirable value for each property listed									
	Low density aggregate; poor curing, causing low moisture retention at high temperatures	Aggregate with low thermal conductivity; dry concrete, air voids	Aggregate with high thermal expansion	Rich mixes; high mixing water	Rich mixes; high-early-strength cement	Rich mixes; low water-cement ratios	Lean mixes	Rich mixes; concrete with high thermal expansion; nonresistant aggregate	Rich mixes high strength concrete	Overvibration causing segregation of high density aggregate; poor control
	Comparison of concretes containing different heavy aggregates									
Iron ore	Good, but give high secondary gamma rays	Good					Some hematite is good	Limonite decomposes at high temperatures		
Steel shot	Good, but give high secondary gamma rays	Excellent						Good		
Barites	Good	Poor	Poor					Weathers poorly		

compression for concrete of varying degrees of saturation, and modulus of elasticity under saturated conditions at elevated temperatures require further investigation. In addition it has been shown that, unless the inside face of the shield is sealed, moisture diffusion and heat diffusion should be considered jointly; the temperature distribution in the shield cannot be found for even steady state conditions unless the moisture condition of the shield is known; shrinkage strains have an important effect on the unrestrained strain distribution.

From the discussion of the influence of various concrete properties on shield behavior, it is possible to review the properties likely to be associated with a concrete that will give good structural behavior. It must possess:

(a) Good attenuation properties, because a thin shield will stress itself no more, and probably less than a thick one

(b) High conductivity to minimize the temperature rise due to attenuation, and to allow a high rate of heat transfer without high temperature differentials

(c) Low coefficient of thermal expansion to reduce the unrestrained strains

(d) Low shrinkage to reduce the unrestrained strains

(e) Low heat of hydration and a low curing growth

(f) Low modulus of elasticity to reduce stresses arising from restrained strains

(g) High creep rate to reduce the stress producing component of the restrained strains

(h) High resistance to radiation and heat damage

(i) High maximum strain at failure

(j) Ample workability to ensure homogeneity

The implications of these requirements regarding mix and aggregate type are set out in Table 4, a study of which indicates that a satisfactory concrete will probably have a clean, clay-free aggregate with high conductivity, high density and low thermal expansion; have a curing period of at least 1 and preferably 3 months under warm moist conditions, have a low cement and water content (which means a lower creep rate but the other advantages outweigh this disadvantage); and be free of segregation and bleeding. In particular, uniform, lean mixes, which have a low shrinkage, can be obtained by the prepacked method of placement in which the aggregate is first packed into the mold, and an intrusion mortar pumped in afterwards.

It is also noteworthy that barite concrete, which has been widely used for shielding because of its good attenuation properties, is poor from the point of view of structural performance because both its low thermal conductivity and its high coefficient of thermal expansion lead to high thermal stresses.

Weathers poorly | Poor | Poor | Good | Barites

ACKNOWLEDGMENTS

The author wishes to thank Prof. J. W. Roderick and Mr. D. Campbell-Allen for much valuable assistance in the course of the work described in this paper.

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Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1961 for publication in the Part 2, December 1961 JOURNAL.

APPENDIX

EFFECT OF PARTIAL DRYING ON THE TEMPERATURE DISTRIBUTION IN A REACTOR SHIELD

Notation

k_1 = conductivity of dry concrete, 1.2 Btu per ft per hr per deg F
k = conductivity of wet concrete, 2.0 Btu per ft per hr per deg F
λ_1 = relaxation length of dry concrete, 0.297 ft
λ = relaxation length of wet concrete, 0.33 ft
x = distance from inside face of shield, ft
h = thickness of shield, ft
$p\,h$ = thickness of dry section of shield, ft
E = average photon energy, Btu
I_o = incident gamma flux, photons per sq ft per hr
θ = temperature rise due to attenuation, deg F

The relaxation lengths and conductivities are based on data by Davis.[1]

Temperature distribution

In the following calculations an examination is made of the possible changes in temperature distribution in a biological shield when it is partially dry. The theory is an approximation as only gamma radiation is considered and no attempt is made to cover transient conditions.

TABLE A1—CALCULATED TEMPERATURES IN A PARTIALLY DRY SHIELD

p	θ at $x = 1$	θ_{avg}
	Deg F	Deg F
0 (wet)	35.8	19.5
0.01	42.8	23.2
0.02	47.2	26.0
0.03	51.5	28.0
0.05	55.7	30.4
0.07	59.0	32.2
0.10	57.2	32.7
0.20	61.5	34.8
1.00 (dry)	64.3	35.8

The hypothetical shield considered is 10 ft thick with an incident gamma flux of 250 Btu per ft per hr and it is also assumed that there is a heat flux through the shield such as to cause a difference of 120 F in the saturated shield.

(*a*) *Saturated shield*—

The basic equation is,

$$k \frac{d^2\theta}{dx^2} = - \frac{EI_o}{\lambda} e^{-x/\lambda}$$

which gives

$$\theta = \frac{EI_o\lambda}{k}(1 - e^{-x/\lambda}) + \frac{EI_o\lambda}{kh} x(e^{-h/\lambda} - 1)$$

substituting numerical values,

$$\theta = \frac{250}{6}\left[\frac{10 - x}{10} - e^{-x}\right]$$

which gives

$$\theta_{max} = 35.8 \text{ F}$$

$$\theta_{avg} = 19.5 \text{ F}$$

(*b*) *Partly dry shield*—

In this case it is assumed that the shield is completely dry to a distance ph from the inside face, and that the rest of the shield is completely saturated. The basic equation for the dry zone is

$$k_1 \frac{d^2\theta}{dx^2} = - \frac{EI_o}{\lambda_1} e^{-x/\lambda_1}$$

and for the wet zone is

$$k \frac{d^2\theta}{dx^2} = - \frac{EI_o}{\lambda} e^{-ph/\lambda_1} e^{-(x-ph)/\lambda}$$

Integrating these and using the boundary conditions at $x = ph$ of continuity of temperature and heat flow gives the temperature in the dry zone as:

$$\theta = - EI_o \left[\frac{\lambda_1}{k_1}(e^{-x/\lambda_1} - 1) + \frac{C_1 x}{k_1 \lambda_1}\right]$$

and in the wet zone as

$$\theta = - \frac{EI_o}{k\lambda}\{\lambda^2 e^{ph/\lambda(1/\lambda - 1/\lambda_1)}(e^{-ph/\lambda} - e^{-x/\lambda}) + C(h - x)\}$$

where C_1 is given by

$$C_1\left(p - (p-1)\frac{k_1}{k}\right) = \frac{\lambda_1}{h}\left[e^{-ph/\lambda_1}\left(\frac{k_1}{k}\lambda - \lambda_1\right) - ke^{h(p/\lambda - p/\lambda_1 - 1/\lambda)} + \lambda_1\right]$$

and

$$C = C_1 \frac{\lambda}{\lambda_1}$$

(*c*) *Calculated temperature rises*—

It was found that in all cases the maximum temperature rise occurred at approximately 1 ft from the inside face, and the temperature rises at this point, together with the average temperature rise is given in Table A1 for various values of P.

CONCRETE BRIEFS

Notes from Field and Office

Nailability of Concrete Slabs 1509
Bond in Flat Slabs 1512
Simple Method for Graphical Determination of Center of Gravity of Trapezoids 1521

Nailability of Concrete Blocks

By A. B. DOVE*

DURING 1956 IN experiments on Haydite block we established that concrete block made of this material should not be nailed to depths greater than one-half of the face wall thickness. Experiments at that time were conducted on nailing with several types of nails in conjunction with tests on Aerocrete, cinder block, and slag block. Since the Cooksville Brick Co. had requested information, we gave them data which would indicate that their block could be nailed to the above-mentioned depth, but that care should be taken to avoid nailing near the edges.

In the "List of Acceptable Materials" published by Central Mortgage and Housing Corp., Canada, the statement is made that Haydite block may be nailed with common, "Ardox," or concrete nails provided that the depth of penetration in the face wall does not exceed half the thickness of that wall.

Tests made by Wheeling Steel Corp. and published in the March 1950 ACI JOURNAL† show the results of nail driving tests in various types of concrete block. The report indicates that cut nails may be driven easily in Slaglite, pumice, cinder, and Celocrete block, whereas common nails may be driven readily only in pumice, while 75 percent or more bent driving in the other types. The information is not clear, but we would assume that these tests were performed by attempting to drive the unsupported nail into the concrete block.

In nail-pulling tests reported in this paper,‡ values of 125 lb in pumice to 250 lb in sand-gravel were indicated for 7d cut nails driven into cell walls. Common nails gave about ⅛ to ⅓ of these values, but could not be driven into sand-gravel or Slaglite.

TEST PROGRAM

The purpose of this present test was to determine nail driveability for various types of concrete block, as well as relative nail holding power with common and spiral nails.

Sample block were obtained with concrete-sand (high and low pressure curing), Haydite (high pressure curing), cinder (low pressure curing), and slag (high and low pressure curing) having 1-in. face walls and with concrete-sand a further sample was checked having a 1½-in. face wall. Nails were driven to various depths through 1-in. pine blocks and the values were compared for 10-gage common, 10½-gage Ardox, and 8-gage heat treated concrete nails.

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†Meder, S. A., "Holding Power of Nails in Concrete Blocks," ACI JOURNAL, V. 21, No. 7. Mar. 1950 (*Proceedings* V. 46), pp. 558-560.

‡Ibid

The method of withdrawal used was the removal of the block by a claw arm attached to a torquemeter reading in in.-lb. The claw was placed under the edge of the block 1 in. from the nail and the 2 in. wide block was pried off the concrete, while the reading was taken on the dial. This method was used rather than direct withdrawal because of the difficulty of restraining full-size concrete block during nail withdrawal in the usual method. Furthermore, it was felt that a prying removal was closer to actual use than direct withdrawal. Using the torquemeter repeated comparisons were made to obtain reasonably average values.

OBSERVATIONS

1. The results obtained in each case are shown in Table 1.

2. *Concrete-sand* block in high pressure and low pressure types were not nailable with common nails or the 10½-gage Ardox nails, due to excessive bending. This result agrees with the Wheeling Steel Corp.'s report in the March 1950 ACI JOURNAL. In the case of these two block a penetration to half the wall thickness by an 8-gage heat treated concrete nail gave values in the order of 100 in.-lb.

3. The results on *Haydite* indicate that it can be nailed with any of the three nails used and that comparable values are provided by the common nails, Ardox nails, and heat treated concrete nails. The concrete nail, because of its large pyramidal point section, or in the case of the Ardox concrete nail, its conical section, do not have as high a holding power at ½-in. penetration as the smaller untreated nails tested. Values of withdrawal loading are somewhat better at greater depths with the Ardox nails since the tendency to spalling on the inside of the wall is reduced by the spiralling action and the nails were able to penetrate further than half the wall thickness and still retain considerable holding power. This same effect was noted on slag block, where the smaller section spiral nail continued to hold to over half the wall thickness—but not more than three-quarters of the wall thickness—without excessive spalling.

4. *Cinder* block gave lower holding power values in almost all cases. They cannot be nailed deeply without breakage or without whole sections cracking away in excessive spalling, not merely as a breakout but as an area. The cinder block was nailable with common nails to a depth of ½ in., with the 10½-gage Ardox to a depth of ⅝ in., but holding power was somewhat lower when a concrete nail was used even though breakage did not occur. The maximum value obtained was 80 in.-lb when nailed ¾ in. into the web using an 8-gage heat treated concrete nail.

5. *High pressure slag* block gave uniform values even to a depth of ¾ in. using the smaller cross-sectioned Ardox nail, whereas ½ in. to possibly ⅝ in. was the greatest depth of nailing for the common nail. Values obtained for the concrete nail at ⅝-in. penetration were somewhat lower.

6. A ½-in. to ⅝-in. penetration with the 10-gage common nail was permissible in the *low pressure slag* block, giving values of 75 in.-lb. At ¾ in. the nail broke through and values commenced to fall rapidly. At even ¾-in. penetration spalling had not occurred with the 10½-gage Ardox nail. Nailed into the web the Ardox nail could achieve a penetration of 1 in. to a value of 120 in.-lb. When the concrete nail was nailed into the web, values as high as 140 in.-lb were obtained. It would appear reasonable to nail with common nails to a depth of only ½ in. or half the face wall thickness, or to about 60 percent of the depth when the smaller cross-sectioned Ardox nail is used and half the face wall thickness when a concrete nail is used.

CONCLUSIONS

1. The above tests lead us to the conclusion that the common nail should not be used with concrete-sand block and that these block should be nailed

TABLE I — TORQUE WITHDRAWAL VALUES

Material	Face wall, in.	10-gage common nail: Penetration, in.	10-gage common nail: Torque withdrawal, in.-lb	10½-gage Ardox nail: Penetration, in.	10½-gage Ardox nail: Torque withdrawal, in.-lb	8-gage concrete heat treated nail: Penetration, in.	8-gage concrete heat treated nail: Torque withdrawal, in.-lb
Concrete-sand (high pressure curing)	1½	Not nailable		Not nailable		½ ¾ 1¼	100 98.3 50.0 (cracked)
Concrete-sand (low pressure curing)	1	Not nailable		Not nailable		½	100 Breakage at other penetrations
Haydite (high pressure curing)	1	½ ⅝ 1	50 95 70 (through)	⅜ ½ ⅝ ¾	10 72 70 80	½ ⅝ ¾	40 70 80 (in web)
Cinder (low pressure curing)	1	½ ⅝ ¾	70 50 (broken) 25 (broken)	½ ⅝ ¾	51.2 70 30	½ ⅝ ¾	15 25 80 (in web)
Slag (high pressure curing)	1	½ ¾	130 150 (broken)	½ ⅝ ¾ ⅞	110 95 112.5 115 (in web)	½ ⅝ ¾	40 50 100 (in web)
Slag (low pressure curing)	1	½ ¾	75 75 (broken)	½ ⅝ ¾ 1 1-1/16	57 110 107 120 (web) 100 (web)	½ ¾	40 140 (in web)

only with properly designed heat treated concrete nails. During such nailing all necessary precautions should be followed regardless of what heat treated nail is employed, and such precautions include the wearing of goggles.

2. Haydite, cinder, and high pressure and low pressure slag block may be nailed with common nails to half the depth of wall thickness. The order of holding powers expected with such a nail is:

(a) High pressure slag
(b) Low pressure slag
(c) Haydite
(d) Cinder

3. The above-mentioned four types of block may be nailed with 10½-gage Ardox nails to depths of as much as 60 percent of the thickness of the face wall and the order of values becomes:

(a) High pressure slag
(b) Low pressure slag
(c) Haydite
(d) Cinder

4. The 8-gage heat treated concrete nails may be used with any of these block and the order of holding power becomes:

(a) High pressure concrete-sand
(b) Low pressure concrete-sand
(c) High pressure slag or low pressure slag
(d) Haydite
(e) Cinder

These values apply up to a depth of ½-in. penetration or half the face wall thickness. When the value exceeds this amount the position of slag block and Haydite is reversed provided the penetration does not exceed 60 percent of the face wall thickness.

5. When nailing into the web with heat treated concrete nails, maximum values are given by concrete-sand block and minimum values by cinder block. The minimum value is still in the order of 80 in.-lb compared with maximum values as high as 140 in.-lb. Such values will, however, vary from

block to block depending on the manufacture.

6. As a general conclusion, from the above experiments it would appear reasonable to state that common nails may be driven in Haydite, cinder, and high or low pressure slag block to ½ to ¾ in. depth depending on the face wall thickness. All the types of concrete block tested from concrete-sand through cinder can be nailed with heat treated concrete nails either of the conventional spiral or fluted types to a depth not exceeding one-half the face wall thickness with safety. Where possible the nail should be driven into the web and satisfactory results will be obtained by ¾-in. penetration of the web. Further penetration is unnecessary and undesirable from the standpoint of possible breakage. Safety precautions, including the wearing of goggles, should therefore be observed.

7. When consideration is given to the nail corrosion aspect, corrosion would be low with the alkaline type block (concrete-sand, Haydite, slag) compared with cinder block. In dry areas, therefore, bright or uncoated nails could be used for the former type. Galvanized coating will retard corrosion of nails in cinder block by the formation of a corrosion product; continual wetting of the nail-block area will increase the corrosion of any nail used in cinder block.

Bond in Flat Slabs

By A. H. S. ANG and C. P. SIESS*

IN A PREVIOUS ARTICLE,† Professor Geer has called attention to the error which may be involved in determining the bond stresses in flat slabs with the use of the bond stress formula for beams,

$$u = \frac{V}{\Sigma o\, jd} \qquad (1)$$

since, in a slab, the shear and the rate of change of moment are not necessarily the same. This conclusion was based on the results of an analysis for a symmetrically loaded circular plate, which indicates that the moment-gradient is significantly greater than the shear in the neighborhood of a circular column. Such an analysis would apply to a reinforced concrete slab only if the reinforcement around a column is assumed to be placed radially with respect to the center of a column as pointed out in the article. In cases of flat plates or flat slabs where the columns are of rectangular cross section and the reinforcement is placed in directions parallel with the column faces, this assumption is far from the actual condition.

The bond formula for beams, Eq. (1), does not apply strictly to the determination of the bond stresses in slabs. Bond is a linear function of the moment gradient, which is the same as the shear at a section only in such members as beams or one-way slabs.

For the circular slab, the relation between moment-gradient and shear may be derived as follows:

Taking the equilibrium of moments of the circular element in Fig. 1a,

*Members American Concrete Institute, Assistant Professor and Professor, respectively, Department of Civil Engineering, University of Illinois, Urbana, Ill.

†Geer, E., "Bond in Flat Slabs," ACI JOURNAL, V. 30, No. 11, May 1959 (*Proceedings* V. 55), pp. 1225-1226.

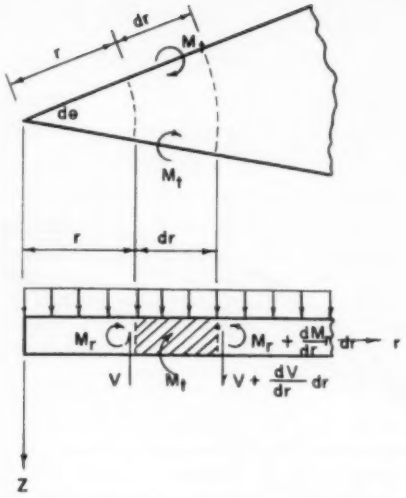
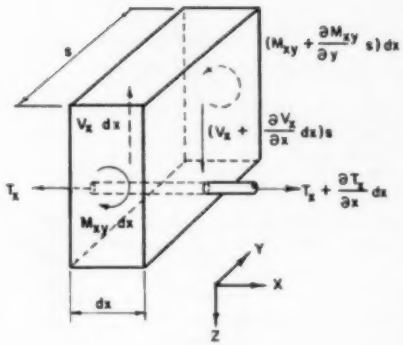


Fig. 1a—Forces acting on a circular element



s = spacing between reinforcing bars.

Fig. 1b—Forces acting on a rectangular element producing moments about y-axis

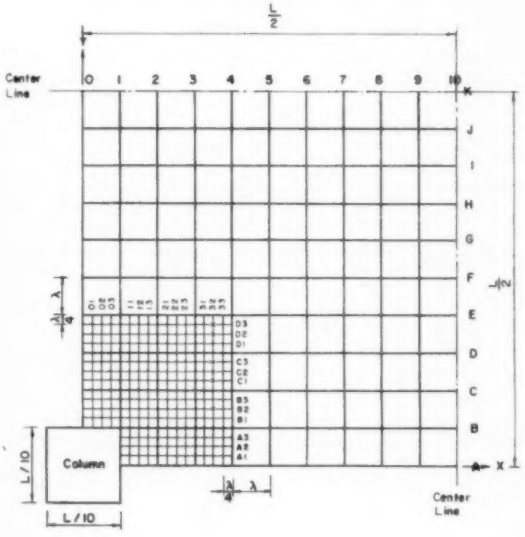


Fig. 2 — Grid sizes on a quarter panel of a square flat plate

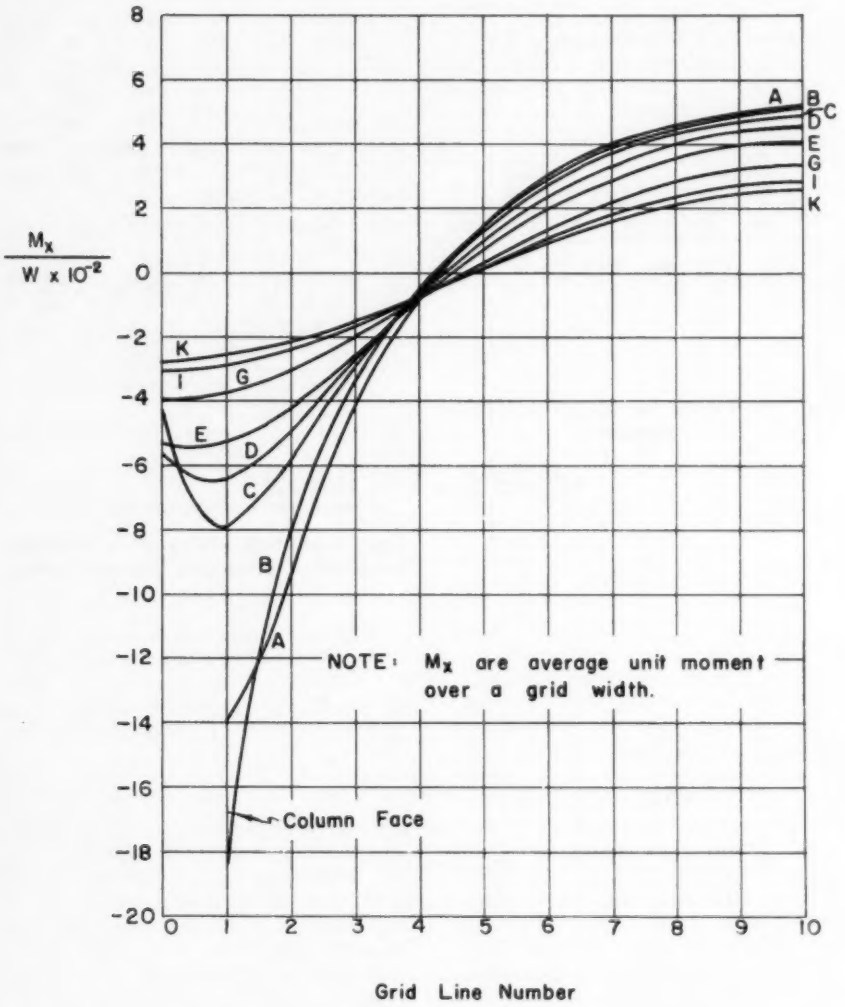


Fig. 3a—Average bending moments M_x along designated grid lines (coarse grid)

$$M_r r d\theta - \left(M_r + \frac{dM_r}{dr} dr \right)(r + dr)d\theta + M_t dr d\theta + V r d\theta dr = 0^*$$

yielding,

$$\frac{dM_r/dr}{V} = \frac{M_r - M_t}{rV} + 1 \quad (2)$$

where M_t is the tangential moment acting on a radial face and the other terms are as defined in Geer's article. The term $(M_r - M_t)/rV$ in Eq. (2) is positive everywhere in the region at and close to the column face, thus resulting in a consistently high ratio of moment-gradient to shear in these regions.

For a rectangular slab, the bond stress and the relation between moment-gradient and shear may be stated, with reference to Fig. 1b, as follows:
The bond stress

$$u_x = \frac{\partial T_x}{\partial x} \cdot \frac{1}{\Sigma o}$$

and

$$\frac{\partial T_x}{\partial x} = \frac{\partial M_x}{\partial x} \cdot \frac{s}{jd}$$

Thus

$$u_x = \frac{\partial M_x}{\partial x} \cdot \frac{s}{\Sigma o j d}$$

But,

$$\frac{\partial M_x}{\partial x} = V_x - \frac{\partial M_{xy}}{\partial y}$$

Thus, the moment gradient in this slab is not equal to the shear alone but is a function of both the shear and the gradient of the twisting moments. The ratio of moment gradient to shear can be expressed as:

$$\frac{\partial M_x/\partial x}{V_x} = \frac{V_x - \partial M_{xy}/\partial y}{V_x}$$

or, since

$$V_x = \frac{\partial M_x}{\partial x} + \frac{\partial M_{xy}}{\partial y}$$

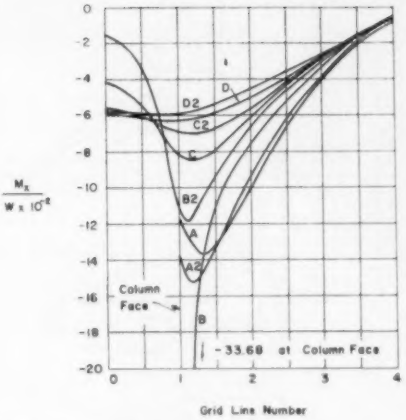


Fig. 3b — Average bending moments M_x along designated grid lines close to column (fine grid)

this ratio can be expressed entirely in terms of the bending and twisting moments as follows:

$$\frac{\partial M_x/\partial x}{V_x} = \frac{\partial M_x/\partial x}{\partial M_x/\partial x + \partial M_{xy}/\partial y} = \frac{1}{1 + \dfrac{\partial M_{xy}/\partial y}{\partial M_x/\partial x}}$$

The results of an elastic analysis of a typical interior panel of a flat plate structure can be used to show the relation between moment-gradient and shear as just derived† The structure analyzed consists of a large number of square panels extending in each direction and loaded with a uniformly distributed load. The columns are square, being one-tenth the panel length, and are also assumed to provide full restraint for the plate. Poisson's ratio was assumed to be zero.

The analysis was made with the calculus of finite differences employing a fine grid of discrete points. At first, a grid, 20 x 20 divisions, was used for

*The sign convention used is the same as in *Theory of Elasticity*, by S. Timoshenko and J. N. Goodier, 2nd Edition, McGraw-Hill Book Co., New York, 1951.

†These analytical studies were made as part of the work being done by the authors under a grant from the National Science Foundation.

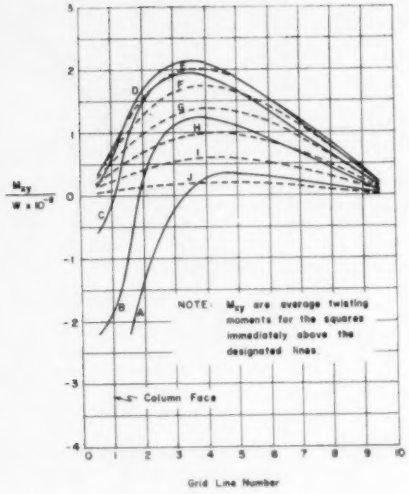

NOTE: M_{xy} are average twisting moments for the squares immediately above the designated lines.
Column Face
Grid Line Number

Fig. 4a—Average twisting moments M_{xy} (coarse grid)

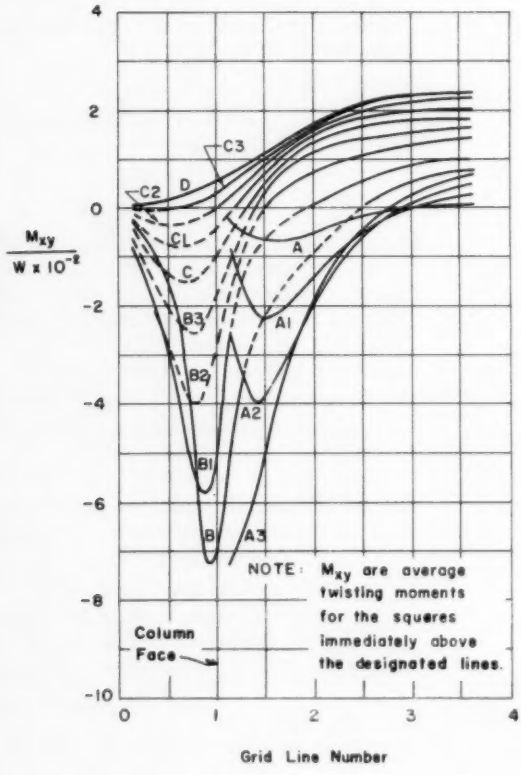


Fig. 4b — Average twisting moments M_{xy} for region around column (fine grid)

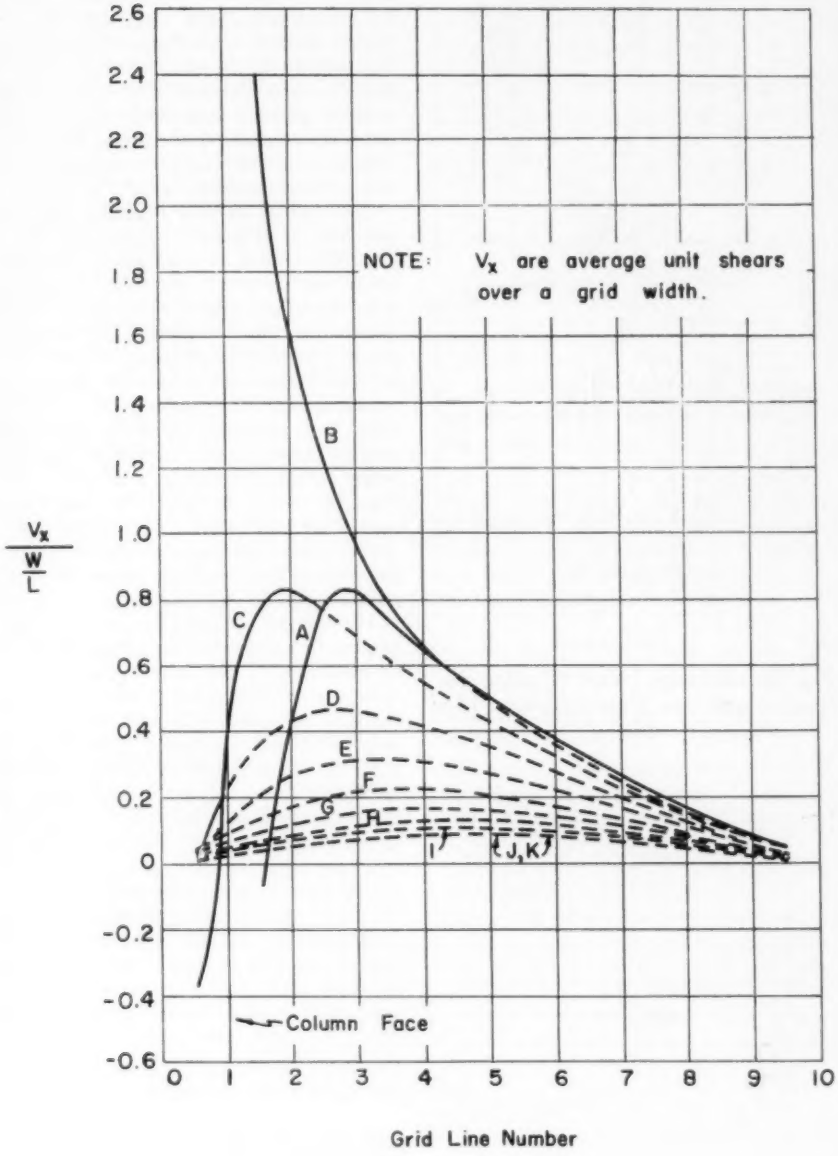


Fig. 5a—Average shears V_x along designated grid lines (coarse grid)

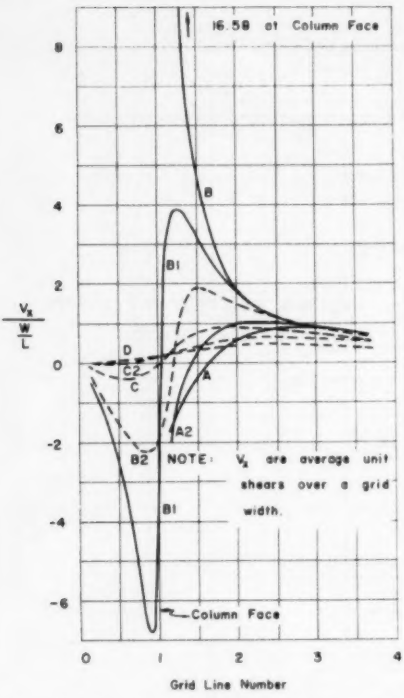


Fig. 5b—Average shears V_x along designated grid lines close to column (fine grid)

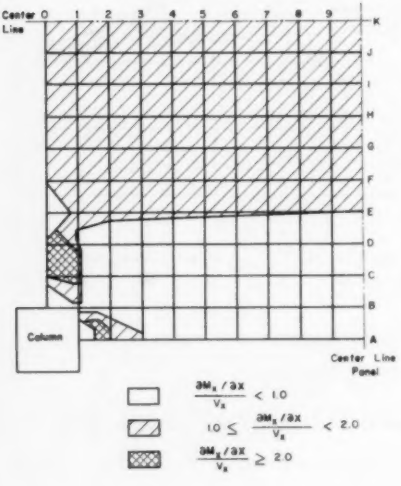

Center Line
Column
Center Line Panel
$\frac{\partial M_x / \partial x}{V_x} < 1.0$
$1.0 \leq \frac{\partial M_x / \partial x}{V_x} < 2.0$
$\frac{\partial M_x / \partial x}{V_x} \geq 2.0$

Fig. 6a (left)—Areas of high and low moment-gradients for quarter panel

the whole panel; because of symmetry, only one-eighth of a panel needed to be considered. Then, the grid in the region around the column was reduced to one-fourth the size of that for the original grid, as shown in Fig. 2. This smaller grid divided each column face into eight equal subdivisions, thus providing a fair estimate of the moments and shears in this region.

The results of this analysis for one quarter of a panel are given in Fig. 3-6. The bending moments, twisting moments, shears, and moment gradients were calculated with the aid of Newmark's plate analog,* and are average quantities over a grid width. The bending moments (see Fig. 3), were calculated at the node points; therefore, the moment gradients are the rate of change of moments between node points. Twisting moments (see Fig. 4), were calculated at the midpoints of the squares between four node points. Shears (see Fig. 5), were calculated for the lines between node points.

The ranges of the ratio of moment-gradient to shear are indicated in Fig. 6 as follows: the absolute† values of the ratio are less than 1.0 in the unshaded regions; between 1.0 and 2.0 in the single-hatched regions; and greater than 2.0 in the double-hatched regions. In some cases the computed ratio of moment-gradient to shear is quite large, even as great as 20 in one case, but the high ratios always occur at points where the shear and moment-gradient are relatively small. In the neighborhood of the column, where the shears are highest, the ratios of moment-gradient to shear are usually less than one. Nevertheless, higher

*Newmark, N. M., "Numerical Methods of Analysis of Bars, Plates and Elastic Bodies," *Numerical Methods of Analysis in Engineering*, Edited by L. E. Grinter, MacMillan Co., New York, 1949.

†Some points have negative ratios of moment-gradient to shear resulting from the reversal of the slope of the moment curves (see Fig. 3b)

values of this ratio do occur near the column where shear and bond may be of importance.

Although the results presented herein are believed to represent accurately the conditions in the plate analyzed, it must be kept in mind that the analysis is for a homogeneous, elastic, isotropic material and involves the usual assumptions of the ordinary theory of bending of plates as well as those introduced by the use of finite differences to obtain numerical values. Moreover, the significance of these values of the ratio of moment-gradient to shear cannot be evaluated without regard to the manner in which the shear is computed in the design of flat plates or flat slabs. In practice shears are seldom if ever computed from the ordinary theory of flexure for plates.

In addition, other phenomena affecting the bond stresses in reinforced concrete structures have not been considered. These include such things as moment redistribution due to cracking, creep, or yielding of the reinforcement, redistribution of bond stresses at a crack, etc. The importance of these factors on the strength and behavior of a reinforced concrete slab can be determined only from tests.

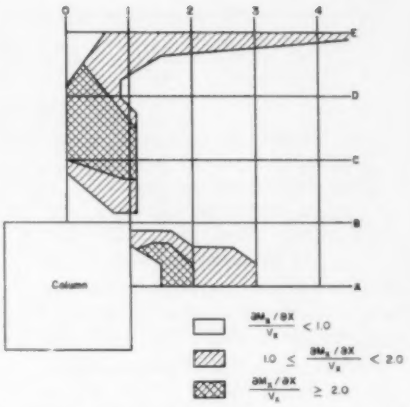


Fig. 6b—Areas of high and low moment-gradients for region around column

By ELIHU GEER*

THE AUTHOR WISHES to express his appreciation to Messrs. Ang and Siess for their contribution to his paper. Their report is a valuable one and adds a great deal to the subject of bond in flat slabs.

During the spring semester of 1959 Gerald L. Paul and Meryle L. Cochrane made and tested a model of a portion of a flat plate to test the author's theory, and to fulfill the requirement for a thesis at the University of Detroit. The test confirmed the computations for one point on the curve in Fig. 3 of the earlier Concrete Brief. The following by-products also resulted:

1. A method was developed for installing electric strain gages for the testing of bond stress, differing considerably from earlier methods.†

2. Because the loads and stresses were low, it was necessary to develop formulas for computing bond stress in the uncracked section.

3. The bond stress in nonradial bars was computed and verified.

The plate was made 6 ft 6 in. square by 4⅛ in. thick, and was placed integrally with a 16 in. diameter stub column (see Fig. 7). The loading was arranged to simulate the condition that $M_r = 0$ at $r = 0.22a$ with the corresponding shear at that location furnished by a ring of weights. The ratio b/a was 0.06 for the test. The bond stress measured on the model was practically identical with the amount

*Member American Concrete Institute, Professor and Chairman, Department of Civil Engineering, University of Detroit, Detroit, Mich., currently Visiting Professor, SEATO Graduate School of Engineering, Bangkok, Thailand.

†Mains, R. M., "Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars," ACI JOURNAL, V. 23, No. 3, Nov. 1951 (*Proceedings* V. 48), pp. 225-252.

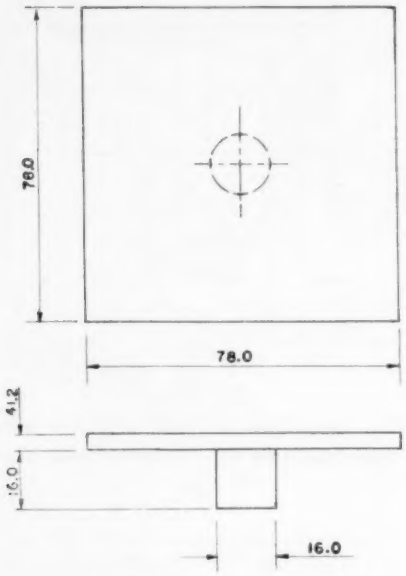


Fig. 7—Test slab

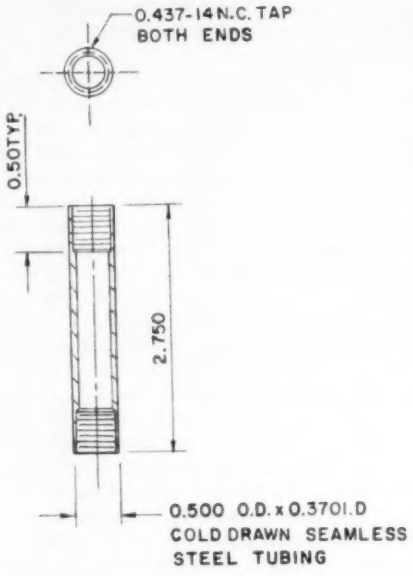


Fig. 8—Strain gage retainer body

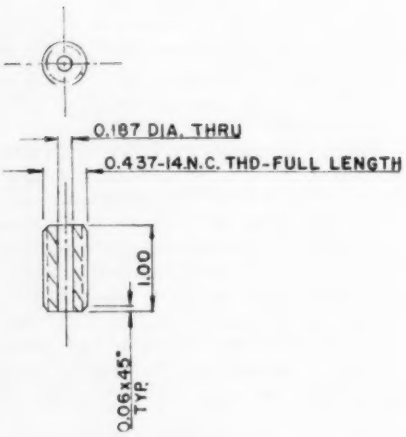


Fig. 9—Retainer body coupling

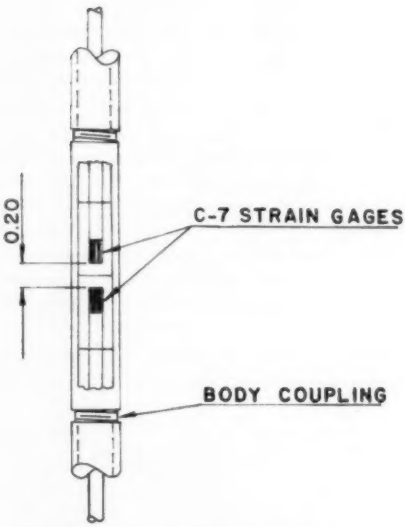


Fig. 10—Test bar assembly

based on the theory presented in the original paper.

The device for determining bond stress is shown in Fig. 8, 9, and 10. Both ends of a short length of tubing were tapped as shown in Fig. 8, to form the *strain gage retainer*. The two gages were cemented within this body at a known spacing. The retainer body was joined by couplings (see Fig. 9) to the tubes which served as reinforcing bars. The assembly is shown in Fig. 10. The time and cost of this device compare favorably with those involved in splitting, grooving, and welding reinforcing bars.

To compute bond stress in an uncracked section it was necessary to first forget the conventional theory and to revert to basic methods. The centroid of the section was located considering the full concrete section and a steel area of $(n-1)A_s$. The moment of inertia (I) of the same section was computed. Then,

$$\frac{dM_r}{dr} = \frac{\Delta TI}{ny\Delta r}$$

where,

ΔT = difference in tensile unit stress in the bar as indicated by the strain gages
Δr = spacing of strain gages
y = distance from centroidal axis to center of bar

As the author's paper raised the question of nonradial bars, it was of interest to find that the simple relationship:

> Bond stress in bar at an angle θ to the radius equals bond stress in radial bar multiplied by the cosine of angle θ

is valid in the case of the circular column. In the vicinity of the column there is no circumferential bending moment (this, too, was verified unintentionally) and the only factor is the extra length of the angling bar.

Simple Method for Graphical Determination of Center of Gravity of Trapezoids

By G. SEGER

Trapezoidal areas occur frequently in design, and the direct utilization of such areas involves the location of the centroid. The graphical method presented here, and translated by ACI member Aron L. Mirsky, originally appeared in VDI Zeitschrift *(Düsseldorf), V. 101, No. 29, Oct. 11, 1959, and is published by permission of VDI-Verlag, GMBH.*—EDITOR

THE CENTER OF GRAVITY of trapezoids is required for many technical problems. With the heretofore-known methods,* only a centroidal axis is projected or the construction lines no longer lie entirely within the trapezoid.

A new and simple method is proposed, which accurately gives the center of gravity of the trapezoid with few lines. The center line *EF* of the trapezoid *ABCD* is divided into three equal portions and thus contains the points *G* and *H* (Fig. 1). The lines drawn from the corners *A* and *B* of the trapezoid through the division points *G* and *H* intersect at point *I*. A line through *I* parallel to the bases *a* and *b* of the trapezoid intersects the center line *EF* in the sought-for center of gravity *S*.

To prove this construction, let the height of the trapezoid be denoted by *h*, the distance $\overline{SI}$ by *z*, the height of the triangle *HIS* measured from point *H* by *u*, and the height of the triangle

*cf, e.g., Föppl, A., *Lectures on Technical Mechanics (Vorlesungen über technische Mechanik)*, V. I, 14th Edition, Munich and Berlin, 1948: especially pp. 123-124.

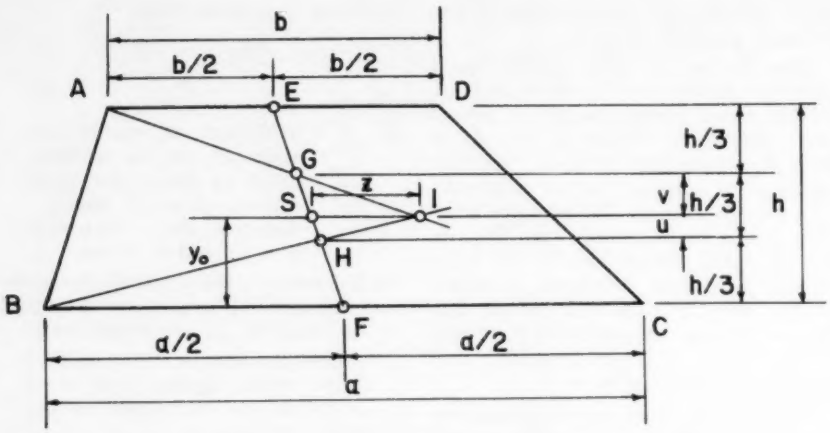


Fig. 1—Graphical determination of center of gravity

GSI measured from point *G* by *v*. Because of the similarity of the triangles *HBF* and *HIS* on the one hand and *GEA* and *GSI* on the other, the relations

$$\frac{\frac{1}{3}h}{\frac{1}{2}a} = \frac{u}{z} \quad \ldots\ldots (1)$$

and

$$\frac{\frac{1}{3}h}{\frac{1}{2}b} = \frac{v}{z} \quad \ldots\ldots (2)$$

are valid.

From these there follow

$$u = \frac{2hz}{3a} \quad \ldots\ldots (3)$$

The Problems and Practices section (which appears in Concrete Briefs section as space allows) is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Perhaps YOUR solution is a better way of handling some everyday problem.

and

$$v = \frac{2hz}{3b} \quad \ldots\ldots (4)$$

as well as, by addition,

$$u + v = \frac{h}{3} = \frac{hz}{3}\left(\frac{2}{a} + \frac{2}{b}\right) \quad (5)$$

and, therefrom,

$$z = \frac{ab}{2(a+b)} \quad \ldots\ldots (6)$$

On placing this expression for z in Eq. (3), there results

$$u = \frac{h}{3}\left(\frac{b}{a+b}\right) \quad \ldots\ldots (7)$$

For the perpendicular distance y_o of the center of gravity measured from the base a there [thus] results the known relation for the location of the center of gravity of a trapezoid,

$$y_o = \frac{h}{3} + u = \frac{h}{3}\left(\frac{a+2b}{a+b}\right) \quad (8)$$

This new method has the advantage that the construction lines always lie inside the trapezoid and [thus] must be a simplification of the methods adduced by A. Föppl.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Elevated road in Ludwigshafen/Rhine (in German)

E. F. WAHL, *Bauverw.*, V. 8, No. 9, 1959, pp. 385-396

ROAD ABSTRACTS
V. 27, No. 6, June 1960

Discusses the traffic problems of Ludwigshafen and the replanning of the city after the Second World War, and gives details of the design of the elevated road which was opened to traffic in July, 1959. The 3000-ft long road is of prestressed concrete construction and is supported by 29 cylindrical concrete pillars.

Deflection surfaces and bending moments of slab-bridge-type plates supported at several points

MUNEAKI KURATA and HIRONAO TAKAHASHI, *Memoirs*, Faculty of Engineering, Osaka City University, Osaka, Japan, V. 1, Dec. 1959, pp. 87-97

AUTHORS' SUMMARY

Deals with the bending of a uniformly loaded slab-bridge-type plate which is freely supported by column head at several points. The number of supporting columns and their arrangement cause occasionally the complex configuration of deflected surface of the plates and interesting features appear. Numerical examples are given for three cases when the square plates are supported at the center point of the plate or at two center points of the free edges or simultaneously at such three points of the previous cases.

Construction

Roof beams post-tensioned on air terminal building

Contractors and Engineers, V. 56, No. 11, Nov. 1959, pp. 58-61

Reviewed by F. F. GAULD

Discusses the roof and other features of the Reno air terminal. The roof consists of ten thin-shell concrete umbrellas, each of which is composed of four hyperbolic sections. These roof shells are 3 in. thick with integral concrete beams running from the center of the marginal beams to the peak of the roof. Precast, tilt-up wall panels form the exterior wall.

Tunnel under Havana Bay (Tunnel sous la baie de La Havane)

J. COURBON, *Annales*, Institut Technique du Batiment et des Travaux Publics (Paris), No. 143, Nov. 1959, pp. 1113-1143

Reviewed by HENRI PERRIN

Describes the whole project which extends for about a mile and includes: a series of ramps and roads connecting the tunnel to the existing streets in Havana; the section of the tunnel constructed in-situ on the Havana side; the underwater section of the tunnel; the section of the tunnel built on the Cabana shore and a large cut on the East Havana side. The 1706 ft long underwater section consists of five precast prestressed concrete boxes fabricated in a drydock and placed in a trench dredged across the channel. The prestressing is of GTM type.

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 11, May 1961, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

The section on the Havana side is a classical reinforced concrete structure, while on the Cabana side a lightly reinforced concrete vaulted structure has been used.

Construction lasted 30 months. Photographs and sketches are included.

Construction Techniques

What every precaster should know about vibration

Concrete Products, V. 63: No. 10, Oct. 1960, pp. 33-36; No. 11, Nov. 1960, pp. 36-40

Part 1 discusses the general methods and techniques presently being employed to vibrate concrete. Part 2 presents some of the practical aspects of vibration as they pertain to the precast producer.

Color in concrete. Part 1—Integral color

Concrete Construction, V. 5, No. 4, Apr. 1960, pp. 90-94

Color on concrete. Part 2—Applied color

Concrete Construction, V. 5, No. 5, May 1960, pp. 132-133

Color may be added to concrete while it is still in the plastic state or after it has completely hardened. Part 1 describes the various techniques for introducing color into the plastic mix; this includes coloring agents affecting the cement paste, decorative aggregates, and combinations of these two.

Part 2 discusses the methods of coloring concrete after it has hardened including the use of paints, stains, plaster, and stucco.

Design

Concepts of structural safety

C. B. Brown, *Proceedings*, ASCE, V. 86, ST12, Dec. 1960, pp. 39-57

Author's Summary

This paper presents methods of providing engineering safety and additional safety necessary for social purposes. The methods considered are the ratio, probability, and combined ratio and probability. Functional and collapse failure are discussed and also the concept of structural life in design.

Some comments on the analysis of skewed orthotropic plates (Einige Bemerkungen zur Berechnung der schiefen orthotropen Platten)

Tibor Javor, *Der Bauingenieur* (Berlin), V. 34, No. 12, Dec. 1959, pp. 474-475

Reviewed by Aron L. Mirsky

Critical comments on methods employed by M. Naruoka and H. Yonezawa (*Der Bauingenieur*, V. 32, No. 10, Oct. 1957, pp. 391-395; "Current Reviews," ACI Journal, Nov. 1958 (*Proceedings* V. 54), p. 660.)

Application of column analogy to the design of shells. Parts I, II, III, and IV

A. Chronowicz, *Civil Engineering and Public Works Review* (London), V. 54: No. 633, Mar. 1959, pp. 326-328; No. 634, Apr. 1959, pp. 477-479; No. 635, May 1959, pp. 623-625; No. 636, June 1959, pp. 763-765

Applied Mechanics Reviews V. 13, No. 10, Oct. 1960

The strength calculations of shells using differential equations is not favored by many engineers for a number of reasons. The author of this paper explains a method not requiring differential equations called "the beam method," applicable to long shells which could be regarded as large beams spanning between traverses with the arched membrane acting as the compression boom while the lower part of the section is transmitting the tension stresses. The method is applicable only to shells with the ratio of the longitudinal span to the chord of the vault greater than 1.5; otherwise the differential-equations method is the only method available. Author shows that the computations could be carried out by the Cross column analogy explained in the book by Cross, Hardy, and Morgan, "Continuous frames in reinforced concrete," Wiley, 1932. Author illustrates the method in a numerical example of a "feather edge" shell where the calculations are arranged in tabular

form and the results are illustrated in numerous graphs.

It must be mentioned that the problem and the method are not adequately explained in this series of articles. Fortunately, the author published at the same time a book on the subject, "The design of shells; A practical approach," London, Crosby-Lockwood & Sons, Ltd., 1959, where in all probability the missing information is contained.

Design of two-way voided lift slabs

JACK STEKETEE, *Consulting Engineer*, V. 15, No. 4, Oct. 1960, pp. 125-126
Reviewed by ARON L. MIRSKY

Six-story housing project for senior citizens at Sylvania, Ohio, is distinguished by voids running along the short direction of the 227 x 53 ft building except at the outer column lines, where they run the long way. Advantages of this two-way over conventional unidirectional voiding included lighter slabs (less load to be lifted), and increased depth and stiffness (hence less waviness). Reinforcement was also two-way.

Stresses in reinforced concrete shields for nuclear reactors

WILLIAM BONSALL, *Proceedings*, Institution of Civil Engineers (London), V. 16 (Session 1959-60), July 1960, pp. 259-270
Reviewed by ARON L. MIRSKY

Analysis of thermal stresses in reinforced concrete slabs with temperature distribution dependent only on distance into the slab reveals that, for slabs constrained to stay flat, a nominal amount of reinforcement to control cracking suffices for temperature differences up to about 50 C, but either the steel or the concrete will be overstressed (beyond 18 or 1.0 ksi, respectively) for temperature differences larger than about 87 C. Use of a thermal shield, with resultant low temperature difference and reinforcement, is thought to be more economical than design for high temperature difference and heavy reinforcement, depending on the main type of radiation causing the heating.

Analysis of shell roofs with a paraboloidical surface

H. C. SHAH, *Indian Concrete Journal* (Bombay), V. 34, No. 8, Aug. 1960, pp. 314-320
AUTHOR'S SUMMARY

Shell roofs curved in one direction have been commonly used in India, but shell roofs curved in two directions have certain advantages and may be expected to find greater use in the near future. This paper gives the mathematical analysis for a general surface curve in two directions and proceeds to apply it to a paraboloidical surface covering a rectangular plane area. An illustrative example of a square paraboloid, 60 x 60 ft, is also worked out in detail.

On the computation of self-supporting winding stairs with intermediate landings (Zur Berechnung freitragender Wendetreppen mit Zwischenpodesten)

H. J. EICHSTAEDT, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 5, May 1960, pp. 117-120
Reviewed by RUDOLPH SZILARD

The mathematical treatment of the problem is based on the theory of indeterminate structures rather than on the theory of elasticity. The stair is subdivided into two statically determinate cantilevers and the redundants are computed by the conventional methods. A numerical example shows the application of the theory.

Practical design of a rectangular floor slab without deepened edge beams (Le calcul pratique d'un plancher a mailles rectangulaires, sans poutre de retombée)

H. PAMELARD, *Le Génie Civil* (Paris), V. 137, No. 8, Apr. 15, 1960, pp. 177-180
Reviewed by ARON L. MIRSKY

Flat slabs supported at the corners, without deepened edges, are analyzed on the basis that the total load in the middle strip is divided into triangular elements by the center lines and the diagonals between the points of intersection of the center lines of the slab bands in one direction and the interior edges of the slab bands in the ortho-

gonal direction. Design of the column capitals, the slab, and the columns is discussed, as is the effect of continuity in the floor.

Analysis of a rectangular plate stiffened by two edge beams (Etude d'une plaque rectangulaire raidie par deux poutres de rive)

Z. HASHIN, *Cahier* No. 7, Cahiers de la recherche-théorique et expérimentale sur les materiaux et les structures, Eyrolles Editeur, Paris, 145 pp.

Reviewed by HENRI PERRIN

This study applies to an elastic material. The first part deals with the theoretical approach: elongations, moments, and internal forces are expressed as functions of the vertical deflections and of the local tensions (Airy's function). The boundary conditions may be solved by an exact method in case of uniformly distributed loadings. The solutions for concentrated loads are cumbersome and the author proposes some approximate solutions based on rapidly convergent series.

The second part describes tests performed on a small scale model. The results agree fairly well with those obtained by the analytical method previously proposed.

Practical stress calculations of circular, cylindrical shell roofs by the theory of beams

KOICHIRO HEKI, *Memoirs*, Faculty of Engineering, Osaka City University, Osaka, Japan, V. 1, Dec. 1959, pp. 73-85

AUTHOR'S SUMMARY

The beam theory of cylindrical shells has already been established. But it has been a troublesome task for a practicing engineer to perform numerical calculations under this assumption, and accuracy of the theory has still been uncertain in spite of some authors' efforts. It could be said the weak point of an approximate theory.

To get rid of the disadvantages, the author introduces first practical expressions for stresses of a cylindrical shell of uniform thickness with edge beams of any form under a symmetrical load. And then, stresses of shells under its own weight, which have some special sections of frequent use, are presented in a simple but more practical form by using diagrams. Secondly, the applicable ranges and errors of this method are discussed referring to results of more rigorous solutions, and then the ranges are extended with some modifying terms.

Vibration of cylindrical shells

KOICHIRO HEKI, *Journal*, Institute of Polytechnics, Osaka City University, Osaka, Japan, Series F, Architecture, V. 1, No. 1, Nov. 1957, 36 pp.

AUTHOR'S SUMMARY

Theories of vibration of cylindrical shell have been worked out by several authors, including Flügge and Naghdi. Those theories, however, are so complicated that, in practice few cases have been amenable to them, except such special cases that the displacement of the shell assumes periodic form in both circumferential and longitudinal directions.

When vibration problems on cylindrical shell roofs are to be dealt with, rather a simplified theory is often desired from the standpoint of practice since actually boundary conditions on the longitudinal edges are far from being simple.

For the purpose of the simplification approximate formulas are presented in this paper under the assumption that only normal component of the acceleration be taken into account while the other components are neglected, and, further, that Zerna's approximations being also employed.

After the accuracy of the present theory being investigated by making use of the characteristic equation, it has been clarified that the formula is applicable to the problem of shell vibration, when the frequency is sufficiently low and the shell is not extremely long. The frequency equation can be solved by the trial-and-error method.

In Part I, the approximate theory thus obtained is applied to the cylindrical shell simply supported on stiff-

ening walls which are rigid in their planes and flexible in the normal direction.

As an example, free oscillations of the shell with free longitudinal edges are solved. On the other hand, experiments have been performed and the results have been compared with the numerical values of the above example, showing good coincidence concerning modes and frequencies.

In Part II the theory is applied to the closed shell, which may have any condition at the circumferential edges, and the main features of the vibration of this type is discussed.

It will be seen from these results that the approximate theory here presented is sufficiently accurate to be used practically.

Analysis of coplanar and spatial piling systems with arbitrary height of pile top and arbitrary pile batter (Die Berechnung ebener und raeumlicher Pfahlsysteme mit beliebiger Hoehenlage der Pfahlkoepfe und beliebiger Pfahlneigung)

KARL SCHNEIDER, *Der Bauingenieur* (Berlin), V. 34, No. 12, Dec. 1959, pp. 462-469
Reviewed by ARON L. MIRSKY

Starting with the general case of pile groups of arbitrary layout, various cases involving successive systematizations of layout are investigated, culminating in the coplanar pile group. Two numerical examples are given, a spatial group with one plane of symmetry and a coplanar group. Basic assumptions are: bearing piles pinned at top and bottom, rigid pile cap connecting tops of piles, effect of soil between top and tip of pile neglected.

Materials

A factual report—Epoxy resins and the concrete industry

W. J. VENEIGH, *Modern Concrete*, V. 24, No. 5, Sept. 1960, pp. 36-41

Summarizes much of the presently known general information relative to the use of epoxy resins. Covers present and future uses and explains some of the advantages of epoxies. Preparations and properties are described including applications as adhesives, fillers, and coatings. Loading, adhesion, and field tests are described with some of the resultant conclusions. In addition to the usual applications some of the lesser known uses are described. A comparison of costs is made and a look at future production. A list of epoxy formulators is appended for those interested in more technical information on the use of epoxy resin compounds.

Corrosion of reinforcing steel in concrete in marine atmospheres

D. A. LEWIS and W. J. COPENHAGEN, *South African Industrial Chemist* (Johannesburg), V. 11, 1957, pp. 207-219
CERAMIC ABSTRACTS
V. 43, No. 9, Sept. 1960

Spalling of the concrete cover around reinforcing steel in the coastal areas of South Africa constitutes a major problem. The severity is influenced mainly by the presence of NaCl in the concrete; other factors include the cement content and water-cement ratio of the mortar. The effects of these variables in the corrosion of the steel are discussed.

Limestone waste as aggregate for structural and no-fines concrete (in Polish)

W. GACA, *Przeglad Budowlany* (Warsaw), No. 7, 1959, pp. 328-330
POLISH TECHNICAL ABSTRACTS
No. 2, (38), 1960

The shortage of stone aggregate in the northern regions of Poland has been remedied by using limestone waste of 50/0 mm fraction. By adding limestone waste to sand, the aggregate is so much improved that it can be used for the preparation of structural concrete grades below 250 kg per sq cm, and of no-fines concrete grades below 80 kg per sq cm. The author discusses the characteristics of limestone waste from the Bielawy quarry and also includes numerical data concerning the physical properties of that stone. The suitability of limestone can be improved by sorting and washing.

Precast Concrete

Neat joints are good business

LAURENCE CAZALY, *Journal*, Prestressed Concrete Institute, V. 5, No. 4, Dec. 1960, pp. 52-53, 56-59

Briefly describes several methods of joining precast concrete elements.

Tetrapods and other precast blocks for breakwaters

P. DANEL, E. CHAPUS, and R. DHAILLE, *Proceedings*, ASCE, V. 86, WW3, Sept. 1960, pp. 1-14
AUTHOR'S SUMMARY

Other shapes have been proposed for precast blocks for breakwaters since tetrapods were first introduced about 10 years ago. The authors briefly consider the problems arising in the design, manufacture, and placing of such blocks.

Consideration is then given to the problem of using precast blocks on breakwaters. Attention is drawn to the great number of factors that have to be taken into account when designing a breakwater. Serious difficulties may be encountered as a result of neglecting or underestimating the importance of any of the problems referred to.

Prestressed Concrete

Considerations about the safety of prestressed concrete structures (Sicherheitsbetrachtungen bei Spannbetonkonstruktionen)

K. KORDINA, *Schweizer Archiv* (Zurich), V. 25, No. 9, Sept. 1959, pp. 319-328
Reviewed by HENRI PERRIN

Presents a short review of the indeterminate factors which affect the computation of internal stresses in a prestressed concrete structure. The fundamental equation for ultimate strength is developed with discussion of each of the indeterminate factors as it is introduced in the design. The ultimate load is determined for simple bending and for bending with compression under static loading. Some considerations about fatigue failure are discussed.

Cast-and-jump builds German bridge

Engineering News-Record, V. 165, No. 20, Nov. 17, 1960, pp. 38-40

Contractor used a novel variation of the cast-and-jump method to build an elevated highway at Andernach, near Bonn, West Germany. Bridge is 1670 ft long, with 13 continuous spans of concrete prestressed both longitudinally and transversely by the Swiss BBRV system. Traveling cranes were used to pull interior truss assemblies into position for casting the next span. Principal advantage of the construction method is the elimination of falsework.

Research on fire resistance of prestressed concrete

HUBERT WOODS, *Proceedings*, ASCE, V. 86, ST11, Nov. 1960, pp. 53-64

Describes the research facilities of the Fire Research Center of the Portland Cement Association. The research program comprises two separate but related parts; there are tests with large furnaces and small-scale basic research on properties of concrete and steel at high temperatures. A long range objective is to assist in the development of reliable predictive methods.

Various aspects of fire resistance research are discussed briefly as well as some work already carried out by other investigators.

Studies on fire resistance of prestressed concrete beams (in Japanese)

IWAWO NISUGI, MICHIUKI KONO, and MISAO SUGAHARA, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 44, No. 9, Sept. 1959, pp. 1-10
Reviewed by KIYOSHI OKADA

Three T-beams, 1 m high, 13 m long, and post-tensioned with four Freyssinet cables of twelve 7-mm diameter wires, were subjected to a temperature of 1120 C for about 2 hr. Heating was done by burning used railroad ties with petroleum oils, simulating actual fire conditions. Conclusion is that the beam strength is reduced after heating due to the damage of concrete but if covering thicker than 5 cm is used it

will protect the prestressing steel sufficiently well from the fire (a mortar layer over 2 cm thick is especially effective for insulation).

The problem of shear strength of prestressed concrete (Zum problem der Schubsicherheit im Spannbeton)

R. WALTHER, *Schweizer Archiv* (Zurich), V. 25, No. 9, Sept. 1959, pp. 329-337
Reviewed by HENRI PERRIN

Proposes a theory about the shear strength of prestressed concrete beams. Equations of equilibrium are written for an assumed shape at rupture and the shear strength is related to the present bending moment. Test results are given to collaborate the theoretical investigation.

Some tests on beams prestressed by fibre-glass cords

STANISLAW KAJFASZ, *Magazine of Concrete Research* (London), V. 12, No. 35, July 1960, pp. 91-98
AUTHOR'S SUMMARY

Discusses tests on 12 concrete beams prestressed by glass fiber cords. The values of the forces in the cables are calculated on the basis of the test results. The influence of bond between tendons and concrete on the behavior of beams is investigated. The low value of modulus of elasticity of glass fiber tendons is discussed.

The results obtained on beams with glass fiber reinforcement are compared with the results obtained on 23 similar beams prestressed with steel wires.

2 and 3-wire strand for prestressed concrete

R. O. KASTEN, *Journal*, Prestressed Concrete Institute, V. 5, No. 4, Dec. 1960, pp. 48-51

Brief description of two- and three-wire strand available for lighter prestressed units such as hollow core machine extruded flat slabs. The 3500 to 8700 psi minimum strengths of these strands, compared with the 9000 to 36,000 psi minimum strengths of common seven-wire strands, offer savings in material as well as material handling costs.

Underground slabs to help anchor piers

RALPH MONSON, *Contractors and Engineers*, V. 57, No. 9, Sept. 1960, pp. 48-49

Unusual prestressing techniques increased plant capacity of this producer. Heavy anchor slabs, buried 13 ft, used compacted earth backfill to resist the pull of the prestressing cables on the piers during pretensioning operations.

Engineering feats with post-tensioning

JEAN MULLER, *Journal*, Prestressed Concrete Institute, V. 5, No. 4, Dec. 1960, pp. 12-40

Reviews some notable long span structures recently completed in Europe and the French Commonwealth. Particular emphasis is placed on features which might be of special interest to the American engineer and contractor.

Prestressed slab has tubular anchorages

Engineering News-Record, V. 165, No. 18, Nov. 3, 1960, pp. 42-44

Unusual application of prestressed concrete pavement at Gatwick Airport, London. Cast-in-place concrete edge strips and unique tubular sleeve anchorages fastened the steel bars used in stressing the slab. The 6½ in. thick, 132 x 150-ft apron was post-tensioned both ways with 1⅛ in. diameter high strength alloy steel bars. Although initially more expensive prestressing eliminated maintenance of contraction joints.

Cylindrical hollow prestressed concrete piles

P. S. HEEREMA, *Journal*, Prestressed Concrete Institute, V. 5, No. 4, Dec. 1960, pp. 41-47

Describes fabrication, handling, testing, and driving of hollow prestressed piles used to support oil drilling platforms. These piles were 36 in. in diameter and had a wall thickness of 5 in. Lengths up to 200 ft were cast in one piece.

Precast, prestressed members trucked 150 miles to site

BILL ALLEN, *Contractors and Engineers*, V. 57, No. 9, Sept. 1960, pp. 27-31

Complex precasting for chemical plant

Contractors and Engineers, V. 57, No. 9, Sept. 1960, pp. 32-34, 39

New casting techniques for prestressed roof beams

Contractors and Engineers, V. 57, No. 9, Sept. 1960, pp. 40-45

Three articles are presented on the $7½ million Parke, Davis and Co. chemical plant, Holland, Mich.

The precast and prestressed members of the structure were fabricated near Chicago and then trucked some 150 miles to the job site. The building is unique in that honeycombed panels form the exterior walls. A typical panel is 18 ft high, 4 ft wide, and 10 in. thick. There are 10,000 plastic pop-out windows. Made of reinforced polyester plastic, the 14 x 20-in. windows occupy 80 percent of the exterior walls of the building. In the event of an explosion these translucent windows pop out of their frames without shattering.

Included, too, are the reasons for the preference of concrete over steel and the use of precast rather than cast-in-place concrete. A novel feature is the introduction of a ½-in. topping of epoxy concrete to resist chemical spattering.

Finally some unusual techniques for casting prestressed roof beams are presented.

Freezing and thawing effects on prestressed concrete

M. J. GUTZWILLER and F. E. MUSLEH, *Proceedings*, ASCE, V. 86, ST10, Oct. 1960, pp. 109-124
AUTHORS' SUMMARY

Freeze and thaw tests were conducted on 48 specimens representing strength levels of 5000 psi and 3000 psi. Some of the beams with a 5000 psi strength were post-tensioned throughout the test. Significant improvement in the durability was found to result from post-tensioning of the beams. Post-tensioned concrete specimens made of a rich mix showed better durability than unstressed concrete made of the same mix and also better durability than unstressed concrete made of a leaner mix.

Prestressed concrete foundation for marine engine test stand (in Swedish)

JIRI SCHMIDL, *Nordisk Betong* (Stockholm), V. 4, No. 3, 1960, pp. 169-188
Reviewed by MARGARET CORBIN

The test stand is an engine foundation, 84 m long on which three maximum size marine engines can be subjected to test runs at the same time, if required. During construction, the test stand was divided by joints into three independent sections, which were stressed together by means of Freyssinet cables in combination with Freyssi flat jacks placed in the joints. When the joints were filled the test stand became an integral unit. Structural details of the foundation are shown. Due to the low speeds of revolution (110-125 rpm) the test stand was designed as a high-tuned foundation. The consequence of this is that the natural frequency of the combined machine and foundation should considerably exceed the impulse frequency (rpm) of the machine itself and exclude all danger of resonance.

Calculation of prestressed concrete members according to the strain theory (in German)

K. KORDINA, *Publication*, International Association for Bridge and Structural Engineering (Zurich), V. 18, 1958, pp. 81-90
APPLIED MECHANICS REVIEWS
V. 13, No. 10, Oct. 1960

This paper is connected with a study by Nik. Dimitrov, "Effects of the strain theory on prestressing," published in IABSE publications, 1956. Dimitrov's conclusions may be cited in brief: "In structures in which prestressing is employed, the deformation in the concrete gives rise, according to theory, to additional second-order permanent moments . . ."

Author disagrees with Dimitrov's conclusions for prestressed concrete

with bond, or even for post-tensioned concrete, where the bond is formed by injection of grout. Author bases his opposition on theoretical considerations, in which the prestressed beam is compared to a concrete frame with a suspended tie rod; and also on the well-known fact that centrally prestressed planks and arches show no buckling effects. Since in practice it is only in exceptional cases that bond of cables to concrete is not formed, Dimitrov's conclusions can be applied to only such exceptional cases.

Influence of prestress on buckling strength (in Dutch)

J. Aarnoudse, *Cement* (Amsterdam), V. 11, No. 4, Aug. 1959, pp. 331-333

Reviewed by John W. T. Van Erp

Conclusions of this study are: (1) Entirely embedded cables have no influence on buckling strength. (2) When the cable is connected to the concrete only at the center, as with exterior cables, the unsupported length cannot be assumed as being halved; this would only be the case with zero external load. (3) If the cable has a certain amount of play in the duct, prestress may theoretically induce buckling.

Marcoule Nuclear Center, prestressed concrete shields for the reactors G2 and G3 (Centre Nucléaire de Marcoule, France, caissons en béton précontraint des réacteurs G2 et G3)

J. Bellier and M. Tourrasse, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 139-140, July-Aug. 1959, pp. 705-736

Reviewed by Henri Perrin

The shields for two reactors of the Marcoule Nuclear Center are made of prestressed concrete. Up to now steel was used as the main structural material while concrete was considered as a shielding material.

The first author describes the planning of the center and the reactors. Some problems of a special nature (each reactor weighs about 82,000 tons) encountered during the design and construction are discussed and well illustrated. Speed of erection and safety were two of the main criteria.

The second part deals with the construction: the schedule was tight and the concrete placement had to be carefully planned. The cables, with a total prestressing force of 2650 kips, were manufactured in a nearby workshop. Setting, adjusting, and tensioning were accurately performed.

Effect of elevated temperatures on high tensile steel wires for prestressed concrete

Maurice F. Day, Eugen A. Jenkinson, and Alastair I. Smith, *Proceedings*, Institution of Civil Engineers (London), V. 16 (Session 1959-60), May 1960, pp. 55-70

Reviewed by Aron L. Mirsky

Four commercially available wires of British manufacture were tested under conditions simulating thermal conditions in prestressed concrete beams in fires. All failed when their temperatures reached approximately 400 C; when heating was discontinued before this rupture temperature was reached, the strain occurring at 300 C was so large that most of the prestress in an actual beam would have been lost, but on cooling many of the tensile properties of the wires showed an actual improvement.

Creep tests at high temperatures, and creep and stress-relaxation tests at room temperatures, brought out markedly different behavior in the four wires tested, depending on the method of manufacture, with the superior performance of one of the wires possibly attributable to a greater capacity for strain aging.

Metallographic data over a range of temperatures are also given.

Effect of variable concrete modulus of elasticity on the deflection of prestressed beams

H. J. Brettle, *Australian Journal of Applied Science* (Melbourne), V. 11, No. 1, Mar. 1960, pp. 92-107

Author's Summary

Author has shown elsewhere that concrete modulus of elasticity E_c increases linearly with prestress. To per-

mit the use of the simple theory of bending it is necessary to transform a rectangular beam section from one of variable E_c to one of constant E_{c1}, resulting in a trapezoidal cross section.

Once cracking commences, the beam cross section and second moment of area are reduced. In the case of prestressed beams, cracks develop slowly owing to the retarding effect of the prestress force. It follows that the second moment of area is no longer a constant after cracking commences, but is a function of both the prestress force and the applied bending moment. In non-prestressed beams, it can be shown that cracks immediately develop almost to their full depth.

The proposed theory is well supported by the experimental evidence from 26 beams of dimensions 4 in. x 6 in. x 10 ft. For a given beam, the variable with the greatest effect on deflection is the prestress force. In the beams tested, the initial pretensioning force ranged from approximately 30 to 300 percent of that normally used and consequently the theory appears valid well beyond the range normally found in practice.

Relaxation, creep, fatigue tests, and tests of behavior at high temperatures of steel wires for prestressed concrete (in French)

D. Jevtic, *RILEM Bulletin* (Paris), No. 4, New Series, Oct. 1959, pp. 66-73

Author's Summary

Reports on relaxation, creep, fatigue tests, and tests of the behavior at high temperature of cold-drawn wire 2.5 and 5 mm in diameter manufactured in Yugoslavia and creep tests on wire of Belgian origin. The results of the tests give relaxation and creep values that correspond to the admissible initial stress values of the steel wires and also give the critical temperature beyond which the drop in strength of the wire is highly accentuated. On the basis of the micrography of the wire it is also possible to estimate the quality of cold-drawn steel wire.

Calculations for prefabricated prestressed monolithic ferro-concrete constructions subjected to a shearing force (in Russian)

S. N. Medvedev, *Sb. Trudi Voronezhsk. Inzh.-Stroit. In-ta*, No. 4, 1958, pp. 39-46; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6880

Applied Mechanics Reviews V. 13, No. 11, Nov. 1960

Fourteen prefabricated monolithic beams of double T and rectangular section were tested, the beams being taken to be hinge-supported, loaded by two forces along thirds of the span. The magnitude of the principal tension forces did not determine the actual moment of the appearance of transverse cracks, while the calculation with the permissible stresses led to an error of 2 to 2.5 times. It is shown that for constructions of the described type a calculation for the shearing force Q on transverse sections can be utilized. Calculations by the formulas of the TsNIIPS give satisfactory agreement with the experimental data; the divergence lies within the limits of from -12 to $+31.7$ percent. The formula for Q is applicable for the calculation of prefabricated monolithic prestressed beams of rectangular section. When making calculations for sections with a flange in the compression zone it is essential to take into account the influence of the flange.

Properties of Concrete

Relation between creep of concrete and the stress-strength ratio

A. M. Neville, *Applied Scientific Research* (The Hague), Section A, V. 9, No. 4, 1960, pp. 285-292

Reviewed by Aron L. Mirsky

Results of recent tests by various investigators are shown to give additional confirmation of the linear relation between creep and stress-strength ratio (ratio of applied stress to strength at time of application of load), the effect of shrinkage (irreversible or drying shrinkage) being minimized by maintaining constant ambient humidity during storage, testing, and recovery periods.

Volume changes of concrete affected by aggregate type

HAROLD ROPER, *Journal*, Research and Development Laboratories, Portland Cement Association, V. 2, No. 3, Sept. 1960, pp. 13-19
AUTHOR'S SUMMARY

Excessive dimensional change phenomena associated with the use of certain aggregates has led to rapid deterioration of concretes in southern Africa, and this paper presents briefly some observations on the effects and probable causes of the distress.

New method for determination of strength of concrete (in German)

T. GYENGO, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 26, No. 1/2, 1959, pp. 103-114
APPLIED MECHANICS REVIEWS
V. 13, No. 11, NOV. 1960

The tensile strength of concrete can be determined generally by two methods. In the first one, test specimens are subjected to pure tension and in the second, unreinforced concrete beams are subjected to bending and tensile strength is deduced from their bending strength. None of these methods is satisfactory and the trouble is that for the determination of tensile and compressive strength, specimens differently shaped and manufactured under different conditions are used. In consequence the two kinds of strengths cannot be considered as related values, characterizing the concrete.

The testing method proposed by Akazawa eliminates the imperfections of the present practice. Tensile strength is determined by applying diametrically opposed compressive loads to a concrete cylinder. Such a loading produces nearly a uniform tension on the section in the plane of loading. Rupture always occurs in the plane of loading when the tensile strength is exceeded. A vertical cylinder of the same dimensions can be used for the determination of the compressive strength. An unambiguous relation between the tensile and the compressive strength of concrete determined in this way. Experiments abroad and in the Hungarian Institute for Building Research proved this new testing method to be correct. It would be advantageous to standardize the test cylinders internationally for the uniform determination of the tensile and the compressive strength of concrete.

Suitability of cement grouts and mortars for injections (L'Injectabilite de Coulis et Mortiers de Ciment)

MICHEL PAPADAKIS, *Revue de Matériaux* (Paris), No. 531, Dec. 1959, pp. 285-302
Reviewed by PHILLIP L. MELVILLE

Study was undertaken to evaluate the principles affecting the rheology of suspensions. As a result "injectability" is defined as the ability of a paste to penetrate the medium to be injected. It is a function of fluidity and stability and is affected by the type fineness and composition of cement and the type, quantity, and gradation of the fine aggregate. Means of checking on "injectability" are given.

Mathematical determination of shrinkage stresses (Die mathematische Erfassung der Spannungen infolge Schwindens)

G. WISCHERS, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 6, June 1960, pp. 273-276
Reviewed by FERDINAND S. ROSTASY

Shrinkage stresses are set up by differential shrinkage of the constituents of concrete or internal restraint of the free shrinkage. There are two kinds of shrinkage stresses besides the ones which are due to restraint in statically indeterminate structures: first, the structural shrinkage stresses which are caused by the restraint of shrinkage of the paste by the nonshrinking aggregate particles. Second, there are the differential shrinkage stresses which are due to the faster drying of the surface layer as compared to the interior of the body.

Author summarizes several theoretical analyses and discusses them. The conclusions drawn are the following: the particle size is of little influence on the structural shrinkage stresses; the higher the modulus of

elasticity of the cement paste is, the greater the shrinkage stresses; differential shrinkage stresses increase with the modulus of elasticity of the concrete and with the free shrinkage; shrinkage stresses are the greater the faster the rate of drying. The beneficial effect of creep is described.

Under water concreting—Behavior of high sulfate, blast furnace and portland cement (Betonnage nous l' eau — Comportamen du Ciment Sursulfate, du Ciment de Haut Fourneau et du Ciment Portland)

LEON BLONDIAU, *Revue de Matériaux* (Paris), No. 532, Jan. 1960, pp. 7-11
Reviewed by PHILLIP L. MELVILLE

Tests were performed to determine strength, density, and porosity. It was found that high sulfate cement is as satisfactory as other cements for under water concreting, has higher strength, lower porosity and higher resistance to corrosion. Concreting was more successful because the mix is more homogeneous and the cement finer.

Measuring the workability of concrete

J. M. PLOWMAN, *The Engineer* (London), V. 209, No. 5447, June 17, 1960, pp. 1007-1009; Discussion Sept. 2, 1960, p. 392
Reviewed by ARON L. MIRSKY

After describing and discussing the shortcomings of various methods proposed (and used) for measuring the workability of concretes, author presents a method suitable for on-site tests of dry, stiff mixes such as are compacted by vibration. Method, based on a suggestion of A. L. L. Baker, measures the amount of work required to compact the mix; it utilizes a sloping channel vibrated vertically at constant power input, with the time required for half a given compacted mass to flow down the channel (with a change in direction at the bottom, as if around reinforcement) onto a balance used as the measured variable. Results of tests of proposed method against comparative methods (slump, compacting factor, and power absorbed in vibrating dry mixes to compaction) are given.

Action of calcium chloride on mortar and concrete

MARGARET LAWRENCE and H. E. VIVIAN, *Australian Journal of Applied Science* (Melbourne), V. 11, No. 4, Dec. 1960, pp. 490-496
AUTHORS' SUMMARY

Severe reaction which causes disruption has been shown to occur when strong calcium chloride solution comes into contact with mortar. The disruption, which is similar to that caused by sulfate attack, is considered to be due to the formation of chloraluminate.

Ultimate loads in rock and concrete

T. N. W. AKROYD, *Engineering* (London), V. 189, No. 4908, May 13, 1960, pp. 672-673
Reviewed by ARON L. MIRSKY

Failure of concrete or rock is a function of the lateral confining pressure; hence triaxial compression tests rather than simple compression tests are necessary. Failure may be due either to the end cones forming and being driven into the test specimen, or to horizontal bursting forces generating vertical cracking. There thus appear to be two schools of thought, one favoring a theory based on cohesion and internal friction (Mohr, Coulomb), and the other favoring a theory based on lateral tension.

Joint testing to determine volumetric mass and blaine specific surface (Essais en Commu de Determination de la Masse Volumique et de la Surface Specifique Blaine)

LOUIS FERET, *Revue de Matériaux* (Paris), No. 529, Oct. 1959, pp. 221-230
Reviewed by PHILLIP L. MELVILLE

A joint testing program was organized between French testing laboratories in the determination of specific gravity and Blaine specific surface. In determining specific gravity by volumeter or pyrometer, it was found that care taken with the actual determination is of greater importance than the method employed. As far as specific surface is concerned, no uniform difference was recorded which could be attributed to either the method employed or the brand of equipment.

When the Blaine test is properly made, results can be reproduced and the fineness of cement determined within 100 sq cm per g.

A stressed state in sections of ferroconcrete elements produced as the result of the shrinkage of the concrete (in Ukrainian)

I. I. Ulitskii, *Vestn. Akad. Str-va i Arkhitekt. URSR*, No. 2, 1957, pp. 26-30; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 6, 1959, Rev. 6900

Applied Mechanics Reviews
V. 13, No. 11, Nov. 1960

Author's data show that because of the influence of the shrinkage of the concrete the stresses in the reinforcement in tension may increase by more than three times. The tensioning stresses in the concrete produced by the external prolonged loading may decrease by more than twice; the compressing stresses, by 20 to 50 percent. The increase with time of the modulus of elastic deformation of the concrete exercises an insignificant influence on the change of the stressed state.

Effect of the type of surface-active agent on spacing factors and surface areas of entrained bubbles in cement pastes

G. M. Bruere, *Australian Journal of Applied Science* (Melbourne), V. 2, No. 2, June 1960, pp. 289-294

Author's Summary

Measurements of the effects on air contents, surface areas of entrained bubbles, and spacing factors in cement pastes were made with a series of pure and commercial surface-active agents, which were used over a range of concentrations.

The results showed that with the agents used, the surface areas of entrained bubbles were markedly different. No correlation was found between the air contents of pastes containing different agents and the surface areas of bubbles.

It is concluded that it should be possible to produce air-entrained concrete with spacing factors adequate for frost resistance, with lower air contents than those now used in practice.

Effect of aggregate size on strength of road concrete (in Russian)

A. M. Boguslavskii and V. F. Osetrov, *Avtom. Dorogi*, V. 22, No. 9, 1959, pp. 8-10

Road Abstracts
V. 27, No. 5, May 1960

Recent research in the USSR is described which indicates that an increase in aggregate size increases compressive strength but reduces flexural strength for the same proportion of cement and water and degree of compaction. According to a graph prepared from data on compressive and flexural strength the most effective aggregate grading is from 5 to 40 mm; a graph based on the coefficient of cohesion and internal friction indicated that optimum grading would be 5 to 20 mm. It is recommended that tests for the required quality of concrete should be based on measurements of flexural rather than compressive strength.

Carbon-dioxide curing of concrete to prevent corrosion by sulfates (in Hungarian)

A. Szekely, *Építöanyag* (Budapest), V. 11, No. 10, 1959, pp. 376-379

Hungarian Technical Abstracts
V. 12, No. 3, 1960

Tests have proved that the method of solidification must also be considered when evaluating the sulfate resistance of concretes since the effect of the method of curing may differ considerably for the various kinds of cement. Treatment with CO_2 significantly improves the resistance of concrete to sulfates. The treatment consists in subjecting the solidifying concrete, after an optimal period of aging, to CO_2 under 2-3 atm gage pressure in an autoclave or by flooding it in an enclosed chamber with CO_2 gas under atmospheric pressure so that the CO_2 content of the air is continually 15-20 percent. The treatment lasts 6 to 8 hr at high pressure and 4 to 7 days at atmospheric pressure. Homogeneous portland cements not resistant to sulfates may be rendered resistant and the resistance to sulfates of other cements may be considerably increased by this treatment.

Structural Research

Limit stress of longitudinal reinforcement of reinforced concrete beams (in German)

K. Heuban, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 26, No. 1/2, 1959, pp. 29-43

Applied Mechanics Reviews V. 13, No. 11, Nov. 1960

Test results obtained from investigating 66 reinforced concrete beams can be summarized as follows:

(a) The limit stress of a longitudinal reinforcement consisting of round steel bars, or Roxor steel bars—the influence of shrinkage excluded—was about 10 percent higher than the lower yield point of the steel bars.

(b) This statement is also valid in the case of a stronger reinforcement, as long as the percentage of steel does not exceed the standard limit, but under that condition the shearing reinforcement and anchorage of steel has to be adequate.

(c) The difference between the two above-mentioned stresses may be explained on the basis of the statistic theory on strength of construction materials, and its calculation is possible under certain conditions. According to this theory, the difference should disappear if the steel is an ideal homogeneous material.

(d) Dispersions appearing in the course of tests on the load carrying capacity of reinforced concrete beams are to a great extent a consequence of the inhomogeneity of the applied reinforcement.

General

Concrete specifications—Their interpretation and administration

Stanton Walker, *The Construction Specifier*, V. 13, No. 5, Oct. 1960, pp. 30-34

Discusses specification writing of ready-mixed concrete as related to manufacturer and consumer.

Author favors prescription type of specification rather than that based on strength alone. Specifications should also reflect minimum cement factor, maximum water ratio, range in slump, and minimum qualities of aggregates with compliance to mixing and elapsed time.

For hot and cold weather concreting author recommends use of two ACI publications: (1) "Recommended Practice for Winter Concreting (ACI 604-58)" and (2) "Recommended Practice for Hot Weather Concreting (ACI 605-59)."

National fire codes, V. 3, Building construction and equipment, 1960-1961

National Fire Protection Association, Boston, 1960, $7

Contains NFPA standards for building construction and equipment which may be applied in a building code. In addition to sections on equipment such as fire doors and windows, incinerators, heating and air conditioning, and restaurant cooking equipment. Structural requirements for building exits, places of outdoor assembly, roofs, chimneys, storage, and construction hazards are included. There is a particularly good section on records management and protection.

Structural concrete

Kurt Billig, St. Martin's Press, New York, 1961, 1020 pp., $17.50

The book has the ambitious aim of being a minor encyclopedia on structural concrete in one volume. While such an objective in a field as broad as concrete is difficult to achieve fully, the author presents a great deal of interesting information on materials, design, and construction. Particularly interesting are the comparisons of design requirements of the American, British, and Russian codes. Part III, Design and Construction, includes foundations, building construction, special structures, marine structures, chimneys, and shell roofs. There are also chapters on radiation shielding, precast concrete, joints and durability.

ACI NEWS LETTER

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memo from the **President:**

Let's be more Consistent and Realistic

Sometimes, even in concrete engineering and construction, a little knowledge may be a dangerous thing. At least it may be dangerous to the extent that it may divert attention from time-tried important aspects of the work. Surprisingly often a new material or a new practice on a job will be automatically named the fault of whatever happens, no matter how grotesque the relationship. Other times a new bit of information, or some test data which may seem indicative, will get far more attention than common aspects of good practice which would be many times more rewarding.

Some years ago, soon after use of air entrainment was started on one job, it so happened that the circular steel forms for the interior of a siphon began to adhere to the concrete and caused some "peeled" surfaces. Project engineers "could not imagine" how air entrainment could cause such a result, but did not look into how well the forms had been oiled or what form oil had been used. When they did, of course, they found that wood form oil had been used and its application was haphazard in the critical areas where concrete entered the forms. Obviously the entrained air had nothing to do with the peeling and it was quite unrealistic to give it a thought.

In some areas there is currently considerable debate and concern as to how certain admixtures and certain elements of cement may affect shrinkage and cracking in concrete struc-

tures. In the first place the matter, particularly the effect of the admixtures, appears to be considerably short of firm resolution, since several reputable agencies in tests of varying, but equally good character, do not arrive at the same conclusion. Nevertheless, a doubt is raised. Not long ago a lively meeting of 200 discussed these matters and prominent men in concrete construction were moved to express concern.

Inconsistency emerges when one stops to think how much more that is positive and considerably greater in effect could be done to minimize shrinkage if the concrete work of their concern was put under effective inspection and control. If excessively wet concrete was rejected, if addition of water to mixers after concrete is first brought to the correct consistency was prohibited, if mixes were composed of materials and proportions which contribute to minimum water requirement, and if curing was ample to develop strength before drying could occur, it could well be that shrinkage factors would be reduced several fold over any possible adverse effects so gravely discussed as being avoided by prohibiting use of these admixtures.

Not only do such jobs routinely lose the value of applied, fundamental, good concrete practices, but they also lose the benefit to be had in many cases from judicious use of the excellent admixtures now available.

Lewis H. Tuthill
President

IN HONOR OF JOE W. KELLY—As he completed his term as ACI president, Joe W. Kelly, professor of civil engineering, University of California, Berkeley, was honored by the structural engineering and structural mechanics divisions of the University. The display presents a thumbnail sketch of Professor Kelly's activity in the concrete field. Included are the ACI Construction Practice Award, the ACI Wason medal, and the gavel presented as a memento of his service as ACI president. One of the books displayed among his writings is the ACI Manual of Concrete Inspection. On the right, the device he developed for making the ball penetration test of concrete consistency, better known as the Kelly Ball Test.

Concrete Progress and Promise Reviewed at 57th Annual Convention

PROGRESS AND PROMISE OF CONCRETE shared the attention at the 57th Annual ACI Convention, February 20-23, in St. Louis. Nearly 700 registered to take part in the committee meetings and technical sessions.

Convention participants were given ample opportunity to see and hear what is new in concrete. They were able to visit the nearly 40 technical committee meetings to learn of the work in progress in as many specialized facets of concrete technology. They were offered for the first time a special session of committee progress which offered more-detailed reports of selected committees and they were given first-hand reports of meetings dealing with concrete overseas.

Two full days were devoted to technical committee meetings and committee activities. In addition to the one and one-half days of technical committee meetings, Tuesday afternoon, February 21, was devoted to a Technical Committee Progress Session. The First General Session was held Wednesday morning, February 22, and was devoted to reports by ACI members on concrete activity overseas.

Sessions on design and construction, and materials were offered concurrently Wednesday afternoon. On Thursday morning, February 23, a Symposium on Fire Resistance sponsored by ACI Committee 216 and a session on design were offered concurrently. Thursday afternoon was devoted to the annual open meeting of ACI Committee 115, Research.

Technical Committee Meetings

Nearly 40 committees or subcommittees held working sessions during the convention. Most of these groups took advantage of the 1½ days set aside for this purpose. There were also administrative meetings of the Board of Direction and the Technical Activities Committee. The TAC Committee on Monographs met as did the chapter representatives.

All of the technical committee meetings were opened to Institute members except in a few cases where committee work had progressed to the point that committees requested a closed or executive session. This important part of Institute work again drew a large number of visitors.

Among the committees meeting were Committee 115, Research; Committee 116, Nomenclature; Committee 201, Durability of Concrete in Service; Committee 207, Mass Concrete; Committee 208, Bond Stress.

Committee 212, Admixtures; Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete; Committee 214, Evaluation of Results of Strength Tests of Concrete; Committee 215, Fatigue of Concrete; Committee 216, Fireproofing or Fire Protection of Structures.

Members of the local St. Louis committee (l. to r.) Paul E. Magoon, exhibits, Carl J. Chappell, executive committee, and Robert A. Willis, secretary

Committee 314, Rigid Frames for Buildings and Bridges; Committee 318, Standard Building Code; Committee 318, Subcommittees 2, 6, 8, and 15; Committee 323 (ACI-ASCE), Prestressed Concrete; Committee 326 (ACI-ASCE), Shear and Diagonal Tension; Committee 331, Structures of Concrete Masonry Units; Committee 332, Recommended Practice for Residential Concrete Work; Committee 333 (ACI-ASCE), Design and Construction of Composite Structures.

Committee 334, Concrete Shell Structures; Committee 336, Combined Footings; Committee 339, Allowable Stresses in Reinforcement; Committee 341, Reinforced Concrete Columns; Committee 401, Specifications for Structural Concrete; Committee 402, Concrete Floor Finishes; Committee 403, Adhesives for Concrete; Committee 505, Design and Construction of Reinforced Concrete Chimneys.

Committee 609, Consolidation of Concrete; Committee 611, Inspection of Concrete; Committee 612, Recommended Practice for Curing Concrete; Committee 613, Recommended Practice for Proportioning Concrete Mixes; Committee 616, Coatings for Concrete.

Committee 622, Formwork for Concretes; Committee 623, Cellular Concretes; Committee 712 (ACI-ASCE), Precast Structural Concrete Design and Construction; Committee 714, Concrete Bins and Silos; Committee 716, High Pressure Steam Curing; Committee 717, Practice in Low-Pressure Steam Curing; and Committee 805, Application of Mortar by Pneumatic Pressure.

Technical Committee Progress Session

As an extension to the 1½ days of technical committee meetings, a new feature was added to the convention program, the Technical Committee Progress Session. This new session featured short reports of current committee activity. These reports were aimed at familiarizing ACI members with committee work of the Institute; helping other technical committees by familiarizing them with the aims, objectives, or problems of these committees; apprising technical committee members of work in other committees which might parallel, overlap, or run counter to the work of their own committee; introducing some of the active Institute members to the general membership, thus giving credit for their behind-the-scenes work toward Institute progress; and suggesting to interested parties opportunities for direct research or sponsorship of research projects where new or additional information is required. The brief résumés were limited to 10-15 min so as to include as many reports as possible. Following each résumé, discussion and suggestions were solicited from the floor. Kenneth D. Cummins, ACI technical director, was chairman of the session.

The following reports were presented:

TAC Committee on Monographs, G. E. Burnett, chairman

Committee 115, Research, S. J. Chamberlin, secretary

Committee 116, Nomenclature, D. L. Bloem, chairman

Committee 212, Admixtures, Bruce E. Foster, chairman

Committee 323, Prestressed Concrete—Joint ACI-ASCE, M. Schupack, chairman

Committee 616, Coatings for Concrete, W. H. Kuenning, chairman

Committee 213, Properties of Lightweight Aggregates and lightweight Aggregate Concrete, J. Neils Thompson

Committee 321, Design of Reinforced Concrete Slabs—Joint ACI-ASCE, C. P. Siess, chairman

Committee 621, Aggregates, R. E. Philleo, secretary

Committee 712, Precast Structural Concrete Design and Construction — Joint ACI-ASCE, Jack R. Janney, chairman

Committee 318, Standard Building Code, Raymond C. Reese, chairman

Concrete Promise

The First General Session, Wednesday morning, February 22, was started on a hopeful note by retiring ACI President Joe W. Kelly, professor of civil engineering, University of California, Berkeley, who recalled to those present that the best years of this unique material, concrete, still lie ahead. Howard A. Coleman, vice-president, Missouri Portland Cement Company, St. Louis, served as session cochairman with President Kelly.

President Kelly described the uniqueness of concrete, a universal material, and of ACI, a cosmopolitan society. "Concrete is not just another commodity;" he said "it has personality." ACI is unique because its membership, he said, "no longer limited to 'cement users,' . . . includes producers and dealers, and individuals in many diverse relations to concrete. It is not a trade association, but brings together the people and interests of many specialized fields to the common ground of concrete. Its members thresh out their common problems within the family." (The complete address can be found on p. 1409 of this month's JOURNAL.

One appeal of concrete is the infinite variety of its application, President Kelly pointed out. One would not think of using wood for a dam, or steel for a pavement, or asphalt for a building frame but concrete is used for each of these and for many other uses of these and other construction materials. Although concrete may not be the principal component of a structure, it is usually found being used for certain portions of the work. It is used to support, to enclose, to surface, and to fill and, he pointed out, more people need to know more about it than about the specialized materials.

While other materials arrive at the construction site as finished products,

Retiring President Joe W. Kelly, left, receives a gavel as a memento of his service to the Institute from his successor Lewis H. Tuthill

Left to right, retiring ACI President Joe W. Kelly, A. Allan Bates, Kennedy Award recipient, and Stanton Walker, Turner Medal awardee

concrete arrives in the form of separate materials which are to be combined by the user. The user therefore must know much about them and their use. Building with concrete requires the combined talents of many people ranging from directors of research to ordinary workmen all working together to produce a final structure.

Concrete will take a lot of abuse, but will sometimes talk back with evidences of maltreatment and, he noted, it is responsive to constructive criticism and unmindful of slander. It ". . . is an ideal—almost limitless in its possibilities for improvement and use; replete with faults frankly acknowledged (and herein lies its strength); and rewarding to those who treat it with proper understanding and care. We fortunate companions who deal with it have a challenge worthy of our best efforts."

One of the unique things about ACI, said President Kelly, was that one member in four lives outside the United States and that many of the members within the United States were born abroad, providing the Institute not only with technical benefits but also a climate for general international understanding. He noted that even within the United States ACI "is committed to the philosophy that cooperation between organizations accomplishes more than narrow competition." He noted ACI cooperation and cordial understanding with the several organizations to which the Institute is officially represented.

President Kelly noted the importance of the work done by the ACI technical committees and he praised the over 600 individuals who give voluntarily of their time in committee work pointing out that while individual committee members are unsung their internal satisfactions are great.

The importance of the new and expanding chapter program was emphasized by President Kelly. He noted that in recognition of this importance the Board of Direction has established a Committee on Chapter Activities and the new Assistant Secretary has chapter matters as one of his major responsibilities.

A concrete that will not crack is the most pressing need of the concrete industry, President Kelly said. A more uniform concrete, uniformity of materials, of mixing and placing, and curing conditions are areas of possible improvement. With more care in workmanship, he said, we can produce far more uniform concretes than we now do. More economical concrete is another need that he pointed out.

We have come a long way, he said "but we can't sit down to wait for the tortoise. The structures of tomorrow will be built largely of concrete and it needs to be better concrete . . . let's look ahead 20 years not 20 years back."

In addition to the President's Address, several interesting reports on concrete activity abroad were presented at the General Session. Douglas McHenry reported on the Sixth Con-

gress of the International Association for Bridge and Structural Engineering, Zurich; Arsham Amirikian reported on the Third International Congress of the Precast Concrete Industry in Stockholm; Eivind Hognestad reported on the Monaco meeting of the Comité Européen du Béton; John K. Selden reported on the RILEM Symposium on Steam Cured Lightweight Concrete held in Gothenburg, Sweden; and A. Allan Bates presented a first-hand report on concrete construction in Russia. All of these reports, except the CEB report, appeared in the April, 1961, News Letter.

An on-the-spot conference for local committee members (l. to r.) August H. Lamack, treasurer, Fred B. Schulenberg, publicity, and A. Carl Weber, executive committee

Design and Construction

Among the projects reported at this concurrent session, Wednesday afternoon, February 22, were structures both unusual and unique employing concrete for both its utilitarian and aesthetic values. The major portion of this session was devoted to concrete put to work—the use of advanced engineering knowledge.

Frank Kerekes, Dean of the Faculty, Michigan College of Mining and Technology, Houghton, was the session chairman.

Full Scale Testing Develops Efficient Preloaded Concrete Pillars — John J. Reed, professor of mining engineering, Colorado School of Mines, Golden; and C. David Mann, St. Joseph Lead Co., Bonne Terre, Mo.

The work described was an effort to develop stronger, more effective concrete pillars for underground mine support. During the past 30 years, concrete pillars up to 12 ft in diameter by 50 ft high have been built occasionally where needed in the southeast Missouri lead mines. In at least two instances, such pillars were built with the object of 100 percent recovery of high grade ore pillars, but generally, and certainly with present costs, these pillars are used only to safeguard important installations. Past experience, substantiated by 20 years of compression measurements on pillars built in 1938, has demonstrated that concrete pillars which are not preloaded will never carry appreciable weight by natural loading until the roof rock and adjacent rock pillars are already failing. In early attempts at preloading, some 16 screw jacks were placed atop the pillars and tightened against the roof. However, their total force was only of the same order as the weight of the pillar itself, about 80 psi on the pillar area, and a much more powerful preloading force was clearly required.

Analysis of data collected during the construction, testing, and preloading of 10 large prestressed concrete mine pillars leads to the following conclusions: 1. Roof over and around the pillar should be pattern bolted, and bolts should be retightened several times to eliminate all possible slack before building pillar. 2. Pillar should be built monolithically of high grade concrete, and prestressed with external banding. 3. A second set of flat jacks should permit high ultimate loading after the first set eliminates excess structural slack. 4. Pillars should have

Checking to see that everything was going according to plan are William A. Maples, ACI secretary-treasurer, and Howard A. Coleman, St. Louis committee chairman

highest possible modulus of elasticity for maximum effectiveness. 5. Actual stope divergence of appreciable magnitude can be obtained by preloaded pillars. Thus, the miner can at last take an active role in ground support. The stope conditions can now be improved rather than simply maintained.

Precast Complex Conoidal Horticultural Domes—W. J. Hufschmidt, president, Hufschmidt Engineering Co., Milwaukee.

A radical departure from the standard gable type roof design for greenhouses or horticultural exposition buildings has been undertaken by the Milwaukee County Park Commission. The project is now well along toward completion at Mitchell Park in Milwaukee.

There are a number of innovations in design, both from the horticultural, as well as the architectural structural, point of view. Mr. Hufschmidt described the design, casting, and erection of one of the typical superstructures of these complex conoidals, or domes, as they are more commonly called.

An interesting aspect was the precasting of the dome sections. The use of concrete molds with loose concrete pieces or cores was selected. Because the large pieces were not in one plane, the molds required loose pieces which had to be released before the finished product could be stripped or removed.

The construction of the concrete mold followed much the same procedure as used in the foundry industry, i.e., a master pattern of the piece was made. It consisted of a number of segments made of plaster of Paris, which were assembled in the final form on an inverted shaped concrete casting pallet. When these several sections had been set perfectly and the intersections joined and molded with plaster of Paris, a perfect finished piece was exposed in an upside down position resting on the concrete pallet. A wood perimeter form was then placed around the finished plaster of Paris and reinforced concrete was cast. This concrete then became the finished mold.

Analysis of Stresses and the Load Carrying Capacity of the Lincoln Boulevard Bridge over Lost River, Idaho—Reynold K. Watkins, professor and head, Mechanical Engineering Department; William A. Cordon, professor; and Elliott Rich, professor, College of Engineering, Utah State University, Logan.

A reinforced concrete bridge was designed during the war to carry normal highway loads. Subsequent developments of the Atomic Energy Commission made it necessary to use this highway to carry approximately twice the load for which the bridge was designed. It was considered necessary, therefore, to replace the bridge or add additional support. A careful analysis of the stresses involved was considered valuable in determining the actual capacity and need for alteration of the original structure.

Tests made to determine the stresses in the steel and concrete under actual field loading conditions were presented and the analysis of the load-carrying capacity of the bridge from these tests given.

It was found the bridge would adequately carry more than double its designed load when all factors used in reinforced concrete design were carefully evaluated.

Supporting Structure for Retractable Roof of Pittsburgh Public Auditorium — Edward Cohen, associate engineer, Ammann and Whitney, New York, and H. Rey Helvenston, resident engineer, Public Auditorium Authority of Pittsburgh and Allegheny County, Pittsburgh.

The unique feature of the auditorium is its vast retractable roof mounted on a circular reinforced concrete ring girder 34 ft above the arena floor. A description of the concrete supporting structure, design conditions, construction of the ring girder and podium, foundation, abutments and anchorages, and the concrete requirements were presented.

Resting on 48 concrete A-frames, the reinforced concrete ring girder supports three sets of tracks on which ride the six movable leaves partially making up the dome of the arena. The reinforced concrete podium deck is of a 10-in. thick concrete slab, about 50 ft wide, that is supported by columns running along the exterior, the center, and the inside perimeter.

All reinforced concrete of the supporting structure was designed in accordance with the ACI Building Code (318-56) including the Ultimate Strength Appendix.

A little before-dinner talk by Waldo G. Bowman, banquet speaker, A. Carl Weber, toastmaster, and ACI vice-president, Roger H. Corbetta

Concrete Materials and Their Uses

The Materials Session, Wednesday afternoon, took a look at something new and something old. Attendants at this session found themselves being asked some penetrating questions on concrete technology.

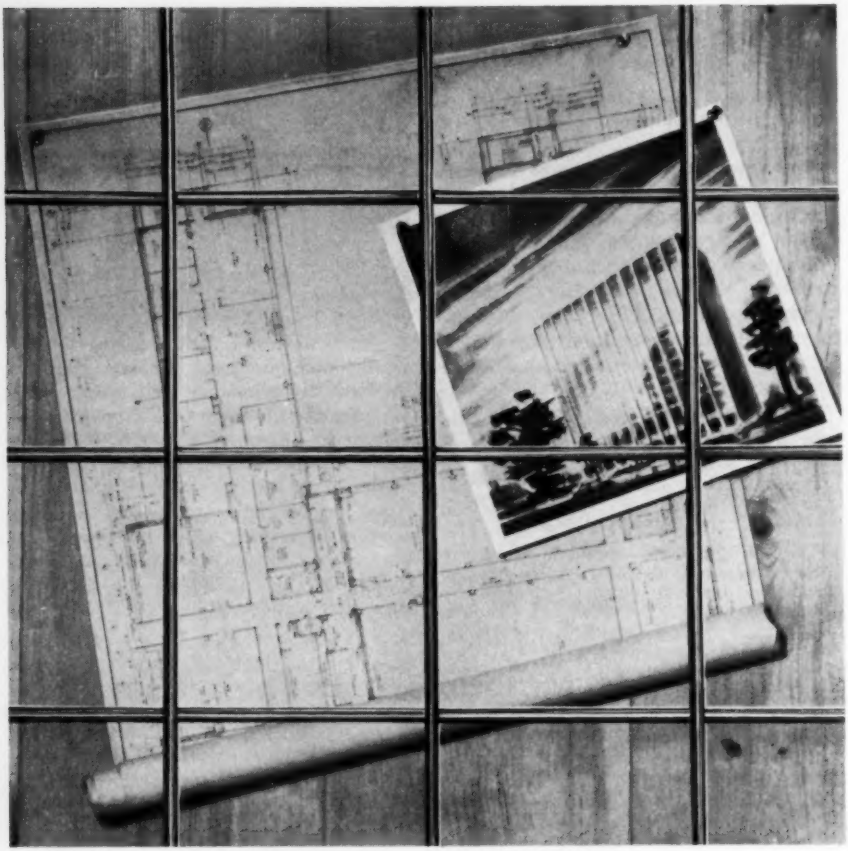
Newly elected ACI President Lewis H. Tuthill, Concrete Engineer, Division of Design and Construction, California Dept. of Water Resources, Sacramento, was session chairman.

Water-Cement Ratio versus Strength — Another Look—Herbert J. Gilkey, professor, Theoretical and Applied Mechanics, Iowa State University, Ames.

Professor Gilkey noted that the water-cement ratio pronouncement marked the most useful and significant advance in the history of concrete technology. However, he pointed out that almost from the beginning there have been protestations from persons who have happened in their tests or researches to touch areas of unusual gradings or areas that entailed comparisons between mortars and concretes or between neat cement pastes and sand-cement mortars.

Besides the actual dissenters there have been thoughtful operators in the area of large-aggregate concrete, used regularly in dams, who, recognizing the lack of information on possible effects of large aggregates and/or large specimens on strength have entertained serious doubts as to whether or not the mass concrete in the structure would develop the strength that the water-cement ratio relationship has allocated to it.

With current attention being redirected toward possible limitations



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in the W/C generalization, Professor Gilkey felt it timely to pull together scattered pertinent evidence that has accumulated over the years; not to discredit the water-cement ratio as a useful empiricism but rather to focus attention on both its range of applicability and on its limitations.

As support for tentative explanations, stress-strain and water-gain data of pertinence were presented. Finally a modified, duly restricted and qualified version of a water-cement ratio versus strength relationship was proposed.

The dominant role of the water-cement ratio as the design criterion for mixtures from similar materials that lie approximately within one general range of proportions, size, and grading was noted. Over any one area (as, for example, that of structural-type mixtures) the water-cement ratio, trial-batch technique maintains its status as unexcelled for simplicity and dependability. For important work the use of the "job curve" as long recommended by the Portland Cement Association continues to provide a workable and highly satisfactory design procedure. "Recognition and acceptance of the qualifying factors discussed and emphasized," he said, "will contribute substantially to the broadening of concrete thinking without any significant alteration in current design techniques over the usual range of concrete practice, while at the same time the informed practitioner stands alerted for taking design account of significant differences heretofore disregarded on the assumption that their influence on strength was nil."

(The paper appeared in the April, 1961, JOURNAL.)

Chemical Prestressing of Concrete Elements Using Expanded Cements — T. Y. Lin, professor, Civil Engineering; Alexander Klein, research engineer; and Tsevi Karby, graduate student, University of California, Berkeley.

Self-stressing produced by an expansive cement was utilized to pre-

Wason Medal Winner Henry Toennies on the left and retiring ACI President Kelly, who made the presentation

stress high tensile steel in tension, thus creating precompression in the concrete. The expansive cement consisted of a portland cement component and a calcium sulfoaluminate component. Four pressure pipes, three beams, two flat slabs, and one thin shell were constructed of concrete, using expansive cement, for prestressing. The behavior and strength of these elements as measured experimentally are compared with analytical values based on conventional theory of elasticity and principles of prestressing.

While many variables need to be investigated to fully utilize the expansive cement for self-stressing, it is clear that practical applications are already possible, on account of the tremendous economy gained through the self-stressing process. Once the

1961
REGIONAL MEETING
November 1-3
Dinkler-Tutwiler Hotel
BIRMINGHAM, ALABAMA

Wilhelm Fuchssteiner, on the left, in the United States from Germany to attend the ACI Committee 336 meeting, compares notes with two of the Institute's newly elected officers, President Lewis H. Tuthill, center, and Director Chester P. Siess

cement becomes available commercially, and if the cost is within limits, its application to the production of pressure pipes and other containers becomes immediately economical. Its use for airport and highway pavements also promises excellent performance and economy. Precast walls and slabs may also utilize such cements.

How Good is Good Enough?—Edward A. Abdun-Nur, consulting engineer, Denver.

How good should concrete be to serve the purpose for which it is intended? To answer this question, several typical specifications were examined and evaluated statistically by relating the results obtained to physical job conditions. The importance of relating test results and statistics to the external physical evidence of the project was stressed.

From this examination and analysis, it is found that: 1. Minimum strength specifications are unrealistic and are not being met in practice. 2. Such specifications tend to obscure the real factor of safety. 3. Specifications that permit the probability of a reasonable number of low strength values in the universe, are more realistic and fit more closely the results being obtained in practice. 4. In turn, this approach lowers costs, reduces maintenance, and permits the factor of safety to be predetermined before construction starts. 5. A probability of 10 to 20 percent of strengths below design strength provides better end-product concrete than the concrete being obtained in current practice under a minimum strength specification.

It was concluded that a requirement for an average strength that provides a probability of about 15 percent of "low" strengths in the universe is "good enough" to answer the question of how good is good enough?

Load Tests of Patterned Concrete Masonry Walls—R. O. Hedstrom, development engineer, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., (Presented by J. J. Shideler).

Described laboratory tests to determine compressive and flexural strengths of concrete block walls laid in nine different patterns. Flexural tests were made on wall panels subjected to bending across a vertical span and also across a horizontal span. Two types of mortar were used. Reinforcing steel was included in some of the walls tested in flexure across a horizontal span. The performance of the walls laid in the various patterns were compared to that of standard running bond.

(The complete paper appeared in the April, 1961, JOURNAL.)

Symposium on Fire Resistance

One of the concurrent sessions, Thursday morning, February 23, was the Symposium on Fire Resistance sponsored by Committee 216, Fireproofing or Fire Protection of Structures, and featured papers by members of that committee. Subjects included fire resistance of reinforced concrete, of building construction protected by concrete, and of prestressed concrete. Committee 216 chairman, Clifton C. Carlson, manager, Fire Research Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., was the chairman of the symposium.

Fire Resistance of Reinforced Concrete—Irwin A. Benjamin, director of research, Granco Steel Products Co., Granite City, Ill.

Available knowledge on the fire endurance of reinforced concrete is reviewed. This included factors affecting the thermal performance, such as properties of the aggregate, and the presence of moisture in the slab. Also discussed were factors affecting the structural performance; the effect of high temperatures on the concrete, strength of the concrete, the cover and volume relationships, tendencies to spall, and the effect of restraint.

Included were data on fire ratings of reinforced concrete based on the ASTM E 119 fire endurance test. Other data showed that the addition of small amounts of plaster can be effectively used to increase the fire resistance ratings of reinforced concrete construction.

Fire Resistance of Building Construction Protected by Concrete—Robert R. Sheridan, structural engineering consultant, Eastman Kodak Co., Rochester, N. Y.

This subcommittee has undertaken to review as much as possible of what has been learned through fire exposure tests of many different concretes and assemblies, to summarize the test results and the more important characteristics of fireproofing materials, and present the substance in brief form.

The subcommittee's general impression is that most of the testing on concrete was done 20 to 50 years ago, that a massive program on 100 columns sponsored by Underwriters Laboratories, Factory Mutual Association, and the Bureau of Standards has never been matched by a comparable program on beams. An important exception to this is the series on solid concrete slabs and beams carried out in 1953 by Underwriters Laboratories for the Portland Cement Association.

Relatively little appears in the JOURNAL about fires and testing for fire resistance. Experiences with fires in two industrial buildings, both of flat slab construction, were described by John G. Ahlers in May, 1930, and by J. Fruchtbaum in January, 1941.

This report contained a brief comment on the general functions of protective materials. It then summarized the effect of high temperatures on structural steel, to identify the limits of practical service. A brief summary of the standard fire test and ratings was given. The body of the report was concerned with steel columns protected by concrete, protection of beams and girders by concrete slab above and plaster ceiling below, and the fire resistance of walls, both cast-in-place and masonry.

Fire Resistance of Prestressed Concrete—G. E. Troxell, professor of civil engineering, University of California, Berkeley.

The code requirements for a given fire resistance of a prestressed concrete structure cannot be the same as for

Local committee members (l. to r.) Bruce Kester, Albert H. Baum, and J. John Brouk

conventional reinforced concrete, as the prestressing steel and the higher quality concrete commonly used in prestressed structures are more adversely affected by a given fire exposure. Many fire tests of prestressed structural elements have been made in England and Holland, and a few have been conducted in the United States and other countries. The available test data reviewed appear to be in reasonable agreement concerning the amount of concrete cover required to protect the steel from the damaging effects of fires of various durations and the precautions necessary to keep the cover in place during a fire. The beneficial effects of insulating plasters and of lightweight concrete in increasing the fire resistance were discussed.

From the factual data given, it can be concluded that prestressed concrete structures are effective in resisting fires lasting up to 4 hr. The principal factor affecting its resistance is the protective cover of the concrete on the steel tendons. However, a secondary condition is the size of the element; the larger it is, the greater its heat capacity and its fire resistance. It was also shown that a given cover of lightweight concrete is about 20 percent more effective than hard rock concrete in developing resistance to fires. The required cover for slabs is less than for beams as the heat of the fire penetrates slabs from the bottom only, whereas in beams it penetrates from the sides as well as from the bottom.

Deflections, Folded Plates, Ultimate Strength Design, and Columns are Design Session Features

A controlled deflection design method, the theory of folded plate structures, the rectangular stress distribution in ultimate strength design, and tie requirements for columns were the topics of this Thursday morning concurrent session. These important design problems were presented by educators and research scientists known to most ACI members and the session was presided over by Past-President Phil M. Ferguson, professor of civil engineering, University of Texas, Austin.

A Controlled Deflection Design Method for Reinforced Concrete Beams and Slabs—Adrian Pauw, professor of civil engineering, and Donald G. Alcock, assistant professor of civil engineering, University of Missouri, Columbia.

In conventional working-stress and ultimate-strength design procedures, beams and slabs are sized for strength and the designer only checks for allowable deflection when he suspects that deflection may be a problem. When lightweight-aggregate concrete is used, deflection is usually more of a problem and the use of the so-called balanced-design method is uneconomical. For such concrete, and in the case where

deflection considerations govern the design of normal-weight concrete beams, the design must then be modified by a trial and error procedure.

The design method described permits the designer to automatically limit the theoretical length-deflection ratio to any predetermined value. This ratio is determined from the flexural rigidity of a transformed cracked section. The computer first calculates a dimensionless Q-factor. This Q-factor is the product of three dimensionless ratios: the ratio of the steel working stress to its elastic modulus, the desired length-deflection ratio and a ratio determined by the type and distribution of the loads. A short-cut procedure was presented for the design of beams and slabs subjected only to uniformly distributed loads.

Theory and Analysis of Prismatic Folded Plate Structures — John E. Goldberg, professor of structural engineering, and K. S. Mahmoud, graduate student, School of Civil Engineering, Purdue University, Lafayette, Ind.

A rigorous theory of prismatic folded plate structures was presented which includes pertinent portions of the theory of elasticity and the classical theory of plates. Using appropriate coefficients derived from elasticity and plate theory, the forces and moments along each edge of each of the constituent slabs are expressed in terms of the displacements at the edges. These formulas are then combined into four equilibrium equations at each joint in much the manner of the slope-deflection method.

After receiving these equations by expanding into Fourier series, solutions of the resulting sets of simultaneous equations are readily obtained by straightforward algebraic procedures. The solutions of these sets of equations establish the magnitudes of the deflections at the joints. Subsequent substitution of these deflections into the formulas for forces and moments at the edges of the individual slabs and into formulas for corresponding loads at interior points permits the calculation of stresses throughout the folded plate.

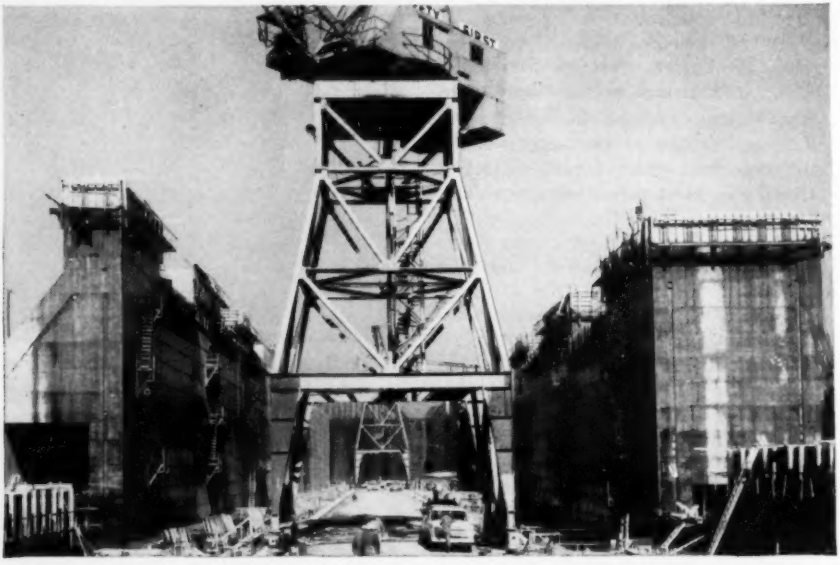
Anton Tedesko, left, Lindau Award recipient, shows the plaque he received to newly elevated Honorary Member Trevel C. Powers

The coefficients, derived by elasticity and plate theory for plate elements, which are used in formulating the joint equilibrium equations have been computed for a wide range of length-to-width ratios. These coefficients are presented in the form of curves, the use of which greatly reduces the effort involved in analyzing a folded plate structure.

Rectangular Concrete Stress Distribution in Ultimate Strength Design—Eivind Hognestad, manager, Structural Development Section; Ladislav B. Kriz, associate development engineer; and Alan H. Mattock, senior development engineer, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

An ultimate strength design theory of broad applicability was developed, based on an equivalent rectangular stress distribution in the concrete compression zone and in general accord with the Appendix to the 1956 ACI Building Code. The theory is characterized by simplicity without significant loss of accuracy.

The proposed method of ultimate strength design was applied to a wide variety of structural concrete beams and columns, subject to various com-



New Ohio River Lock Demonstrates Important Advantages of Fly Ash Concrete

The many benefits of Fly Ash in structural concrete are well demonstrated in the project illustrated above.

The new Ohio River Lock at Louisville, Kentucky, built by the U. S. Army Corps of Engineers, called for concrete to meet exacting tests at every point.

Approximately 390,000 cubic yards of concrete were used in the lock itself and in connecting structures housing the operating and controlling machinery and providing tunnels for discharge of water. All of this concrete contained high quality fly ash from a local source.

This concrete must literally "hold water." Most of it is constantly in contact with river or weather. Its high resistance to penetration by water is due in large part to the tendency of Fly Ash to eliminate voids and to combine with free lime, forming additional insoluble cementitious materials. This pozzolanic process goes on for many months, thus increasing the ultimate strength and serviceability of the entire structure.

The ball-bearing shaped particles of fly ash increase workability of concrete resulting in greater ease and speed of placement. This increased workability is also an aid in forming sharp corners and an aid to finishing.

Each of the companies below has technical data and competent engineers to help you in designing the most effective mixes employing cement and Fly Ash. We invite your call for more specific information.

WALTER N. HANDY COMPANY, INC.
P. O. Box 549, Evanston, Ill.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

WEST PENN POWER COMPANY
Cabin Hill, Greensburg, Pa.

McNEIL BROTHERS, INC.
P. O. Box 4015, Bridgeport 7, Conn.

CHICAGO FLY ASH COMPANY
228 N. La Salle St., Chicago 1, Ill.

binations of bending and axial load. Calculated ultimate strengths were compared with experimentally determined ultimate strengths for a wide range of variables, and an excellent agreement resulted.

It was concluded that the proposed extension of the rectangular stress distribution theory permits prediction with sufficient accuracy of the ultimate strength in bending and compression of all types of structural concrete sections likely to be encountered in structural design practice, including odd-shaped sections and other unusual cases.

(The complete paper was published in the February, 1961, JOURNAL.)

Tie Requirements for Reinforced Concrete Columns—Boris Bresler, professor of civil engineering, and P. Gilbert, Department of Civil Engineering, University of California, Berkeley.

Mechanism of failure of reinforced concrete columns with lateral ties was described. Rational criteria for spacing and size of lateral ties were derived, and pilot tests verifying these criteria described.

Previous tests indicated that reinforced concrete columns with spiral lateral reinforcement possess greater deformability at ultimate failure than do similar columns with lateral ties. The mechanism of failure of columns with spiral reinforcement was described by numerous investigators and a rational analysis of the effect of a lateral spiral on the behavior of columns has been formulated. This analysis led to the development of rational criteria for design of spirals.

The authors considered the effects of lateral ties on the behavior of reinforced columns, and derived analytical expressions for the spacing and size of lateral ties.

1961 OFFICERS

At the Awards Luncheon of the 57th Annual Convention, Wednesday, February 22, the newly elected ACI officers were installed. They were:

Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento—president

Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York and Chicago—vice-president

Arthur R. Anderson, partner, Anderson, Birkeland, and Anderson, Structural Engineers, Tacoma, Wash.; **G. E. Burnett,** assistant chief research engineer, U. S. Bureau of Reclamation, Denver; **Chester P. Siess,** professor of civil engineering, University of Illinois, Urbana; and **Anton Tedesko,** vice-president, Roberts and Schaefer Co., New York—directors.

Bryant Mather, supervisory civil engineer, Concrete Research, U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Jackson, Miss., was appointed by the incoming Board of Direction to fill the vacancy left by Mr. Corbetta's election to vice-president.

For a complete story of the 1961 ACI officers see the April, 1961, JOURNAL, News Letter section.

Annual Research Session

Closing out the technical program of the 57th Convention was the Annual Research Session. This session sponsored by ACI Committee 115, Research, as part of its work of reviewing and correlating research in concrete and reinforced concrete brings to the convention participants brief reports of research in progress not sufficiently advanced

AWARDS

At the Awards Luncheon of the 57th Annual Convention, Wednesday, February 22, Joe W. Kelly, retiring president, presented the ACI Annual Awards in behalf of the Board of Direction of the Institute.

Turner Medal — The Turner Medal was awarded to **Stanton Walker,** director of engineering, National Sand and Gravel Association and the National Ready-Mixed Concrete Association, Washington, D. C.

Lindau Award — The Lindau Award was presented to **Anton Tedesko,** vice-president, Roberts and Schaefer Co., New York.

Kennedy Award — The Kennedy Award was presented to **A. Allan Bates,** vice-president, Portland Cement Association, Chicago.

Wason Medal for Research — The Wason Medal for Research was presented to **Henry Toennies,** assistant director of engineering, National Concrete Masonry Association, Washington D. C., for his February 1960 ACI JOURNAL paper.

Wason Medal for Most Meritorious Paper — The Wason Medal for the Most Meritorious Paper was awarded to **Robert A. Williamson,** chief structural engineer, Holmes and Narver, Inc., Los Angeles, for his May 1960 ACI JOURNAL paper.

Honorary Member—Honorary membership was conferred on **Treval C. Powers,** research counselor, Research and Development Division, Portland Cement Association, Skokie, Ill.

The complete awards story appeared in the April, 1961, JOURNAL, News Letter section.

to allow a formal published report. Chester P. Siess, professor of civil engineering, University of Illinois, Urbana, Ill., is chairman of the committee and S. J. Chamberlin, professor, Theoretical and Applied Mechanics Laboratory, Iowa State University, Ames, is secretary.

This annual open meeting of Committee 115 differs from general sessions in that the information is not released for publication. Attendance at the session was the only way of being informed of the developments reported.

Reports of the following research were presented:

Influence of Cement Composition on the Effectiveness of Water Reducing-Retarding Admixtures—Howard Newlon, Jr., Virginia Council of Highway Investigation and Research, Charlottesville.

Extremely Lean Mass Concrete with Pozzolans—Thomas B. Kennedy, U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss.

Mechanism of Fatigue Failure in Concrete—John W. Murdock, research assistant, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana.

Dynamic Behavior of Continuously Reinforced Concrete Slabs—A. D. M. Lewis, School of Civil Engineering, Purdue University, Lafayette, Ind.

Behavior of Reinforced Concrete Frames Subjected to Repeated Reversible Loads—V. Bertero, associate professor of civil engineering, University of California, Berkeley.

Mechanism of Tension Cracking in Reinforced Concrete—Bengt B. Broms, School of Civil Engineering, Cornell University, Ithaca, N. Y.

Ultimate Strength of Reinforced Concrete Beams in Combined Torsion, Bending, and Shear—Hans Gesund, associate professor of civil engineering, University of Kentucky, Lexington.

Effect of Shrinkage Cracking on Diagonal Tension Strength—J. Neils Thompson, professor of civil engineering, University of Texas, Austin.

Prestressed Concrete with Fiber Glass Tendons—David P. Billington, associate professor of civil engineering, Princeton University, Princeton, N. J.

Protection of Steel Reinforcement from Corrosion in Concrete Contaminated with Salt—Charles H. Scholer, professor of applied mechanics, Kansas State University, Manhattan.

Research compilation

Another important part of the work of Committee 115 is the annual Compilation of Research in Progress by various agencies throughout the world. Each convention participant attending the research session was given a copy of this report. Copies are available from ACI headquarters for those who were unable to attend the convention.

The 1961 compilation lists more than 800 projects in plain and reinforced concrete. Over 120 organizations are included. Of this number 45 were United States academic institutions, 33 were United States nonacademic institutions, and 43 were institutions outside the United States.

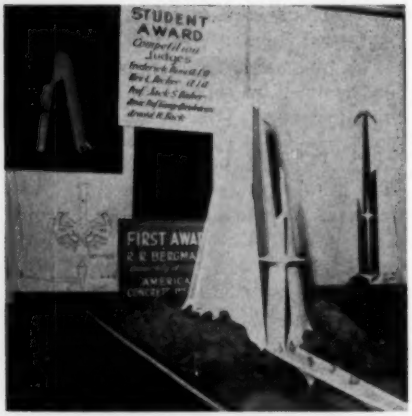
Countries other than the United States reporting this year were: Australia, Canada, Denmark, England, France, Germany, India, Italy, Japan, New Zealand, Norway, Portugal, Scotland, South Africa, Sweden, Switzerland, and Wales.

St. Louis Registration Records

Converging from 40 states, the District of Columbia, Canada, Germany, Mexico, Nicaragua, and the Union of South Africa, 684 registrants attended the ACI 57th annual convention.

Following are registrations from the various states and countries:

Missouri	150	Mississippi	4
Illinois	144	Washington	4
New York	46	Arizona	3
Ohio	34	Louisiana	3
District of Columbia	26	North Carolina	3
Colorado	25	Tennessee	3
California	24	Arkansas	2
Michigan	24	Minnesota	2
New Jersey	24	Montana	2
Pennsylvania	22	Oklahoma	2
Canada	20	West Virginia	2
Texas	17	Connecticut	1
Iowa	16	Delaware	1
Indiana	15	Kentucky	1
Wisconsin	12	Maine	1
Virginia	9	Maryland	1
Kansas	6	New Mexico	1
Nebraska	6	Oregon	1
Georgia	5	Utah	1
South Carolina	5	Germany	1
Alabama	4	Mexico	1
Florida	4	Nicaragua	1
Massachusetts	4	Union of South Africa	1



FIRST PRIZE—Richard R. Bergmann

SECOND PRIZE—George A. Albers

THIRD PRIZE—Richard A. Kimbrough

Architectural Competition

A special feature of the convention was an Architectural Competition sponsored by the local committee. The objective of the competition was to recognize and reward significant contributions of architectural students in the design of structures through a more efficient use of concrete. To be eligible a student had to be enrolled in his fourth or fifth year in a recognized course of study of architecture at Washington University, St. Louis, or the University of Illinois, Urbana, and had to have taken part in the design and planning of the design submitted.

The students were required to submit their designs describing the structure by sketches, plans, sections, elevations (or renderings), and a model.

(Continued on p. 37)

Entries to the design competition. The project was to design a concrete chapel with a seating capacity of 400 persons for an island naval base

Convention Exhibitors

A wealth of educational material awaited convention-goers in the Chase exhibit hall. Twenty-six companies and organizations serving the concrete industry displayed their product lines and services. Technical information and advice were readily available from company representatives manning the booths and in technical pamphlets and product literature. The line-up of exhibitors included:

American Steel & Wire Division of United States Steel, Cleveland, O. — The accent was on American welded wire fabric for all types of concrete slab construction. Booth visitors also learned that the welded wire fabric is now available with extra-heavy ½ in. diameter wires spaced as close as 2 in. on centers in both directions. Descriptive literature was available.

Autolene Lubricants Co., Denver, Colo.—The uses of Protex dispersing agent and Protex air-entraining solution were illustrated in color slide display. Available literature also described Protex curing, sealing, and bonding products; Probond epoxy formulated products, and other concrete additives.

Calcium Chloride Institute, Washington, D. C. — Display and literature pointed up use of calcium chloride for concrete construction. Among the helpful literature were folders on how to prepare standard calcium chloride solution, amount of solution per batch, and data on automatic dispensers.

Chicago Fly Ash Co., Chicago, Ill.—The advantages of using fly ash in concrete were well documented in color slides and literature. Handout literature included proportioning guides for concrete mixes containing fly ash.

Clipper Manufacturing Co., Kansas City, Mo.—On display was the Clipper line of masonry and concrete cutting saws and blades. Also distributed to visitors was data material on Clipper diamond core bits and accessories for concrete and masonry drilling.

Contractors and Engineers Magazine, New York, N. Y. — The Buttenheim Publishing Corp. sponsored an interesting display concerning its monthly construction publication.

Engineering News-Record, New York, N. Y.—Color slides and literature pointed up the construction news coverage of EN-R, weekly engineering magazine issued by the McGraw Hill Publishing Co.

Expanded Shale Clay and Slate Institute, Washington, D. C.—This display featured the many aggregates produced by Institute members. Technical data of value to the concrete engineer and designer was available.

Granco Steel Products, Co., St. Louis, Mo.—Information and descriptive brochures were available to those inquiring about Cofar, a galvanized steel form for concrete which becomes the main reinforcement after concrete has set. Another product manual covered Tufcor, a corrugated steel roof deck which is overlaid with an insulating lightweight concrete.

Johns-Manville Products Corp., New York, N. Y.—The control of concrete through the use of additives was the theme of J-M's Celite Division exhibit. Display highlighted Placewell, water-reducing and dispersing admixture, and Retardwel, liquid retarder for concrete.

Laclede Steel Co., St. Louis, Mo.—Visitors learned about the new size and strength markings on Laclede Multirib reinforcing bars. This positive identification, it was said, will facilitate construction and eliminate inspection and laboratory test charges.

Leschen Wire Rope Division, H. K. Porter Co., Inc., St. Louis, Mo.—Display called attention to the Leschen stress relieved strand for prestressed con-

crete. Visitors were given a booklet describing a wide range of prestressed applications, methods of producing prestressed concrete members, and useful technical data involving post-tensioning, pretensioning, and a combination of both these methods.

Maginniss Power Tool Co., Mansfield, O.—The Maginniss screed for vibrating wide pavements was on display, along with lightweight motor-in-head vibrators. Also on exhibit were vibrator attachments and generators.

The Master Builders Co., Division of American-Marietta Co., Cleveland, O. —Photographs and reports of outstanding structures where Pozzolith was used featured this exhibit. Literature was distributed on Embeco grout, mortar, and concrete, and Master Builders' new concrete research and technical center.

In the exhibit hall, convention participants were able to see a variety of developments and services that they might later apply to their work. The montage above represents a few of the booths that visitors to the exhibit hall were able to examine

Missouri Association of Prestressed Concrete, St. Louis, Mo.—The advantages of specifying precast prestressed concrete structural units for concrete construction were pointed up by this exhibit. Some of the advantages enumerated through illustrations and literature included long clear spans, shallow structural depths, deflection control, etc.

Portland Cement Association, Chicago, Ill.—A mock-up of a hyperbolic paraboloid umbrella shell was an attention-getter and served to emphasize the unusual shapes possible in concrete construction. Display, which also featured color slides of outstanding structures, was sponsored by the St. Louis office of PCA.

The Prescon Corp., Corpus Christi, Tex.—Color slides and literature depicted a wide variety of structures which use the Prescon system of prestressing. Developed in 1950, the system employs the post-tensioning technique and utilizes cold formed heads on high-carbon steel-wire tendons to provide positive end anchorage.

Rex-Spannall, Inc., New York, N. Y. —A nationwide specialist in the concrete forming and shoring field, the Rex-Spanall exhibit emphasized its horizontal shoring techniques and equipment for building and heavy construction. Brochure detailed the Rex-Spanall line of equipment and engineering services.

Joseph T. Ryerson & Son, Inc., Chicago, Ill.—The use of Ryerson post-tensioning materials and engineering services for prestressed concrete construction was spotlighted. Another feature was a description of the lift slab (Youtz-Slitz) method of construction, a specialty of the Skyhook Lift Slab Corp., Kansas City, Mo.

Shapco-Sodemann Heat & Power Co., St. Louis, Mo.—The Shapco atmospheric cabinet, equipped for both dry and submerged storage of test specimens, was on display. The cabinet is built to meet the requirements of laboratories for research or production testing of concrete, with automatic control of temperature and relative humidity.

Sika Chemical Corp., Passaic, N. J.—Sika's wide assortment of concrete and masonry compounds was in the limelight: concrete admixtures, joint sealants and coatings, surface retardants, epoxies, etc. Applications and technical data were detailed in "case history" literature.

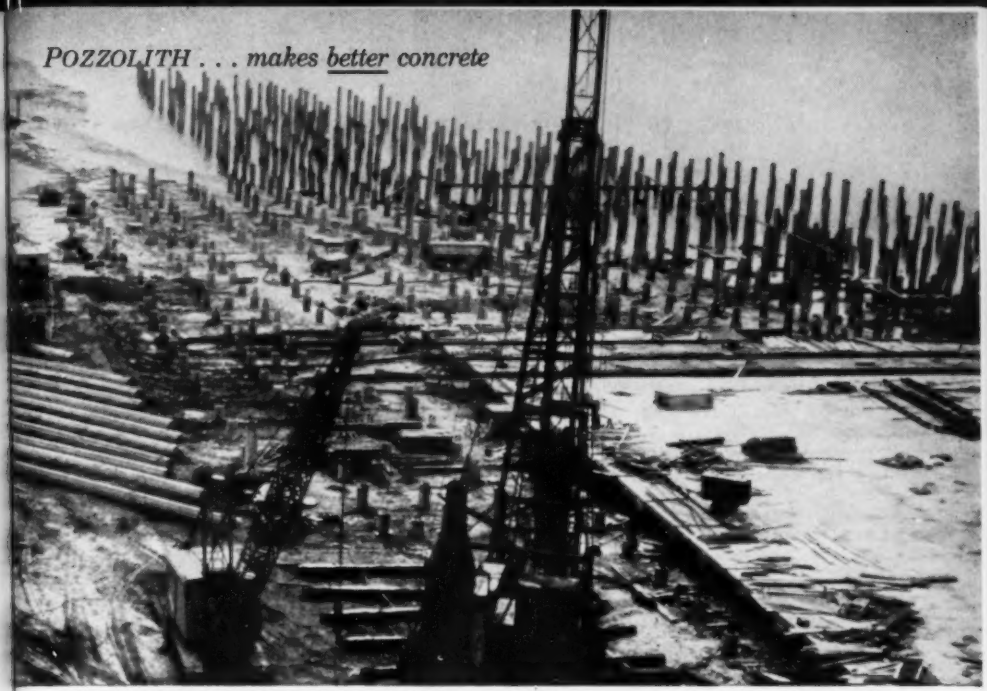
Sonoco Products Co., Hartsville, S. C. —This exhibit featured a pictorial record of concrete construction projects in which Sonovoid and Sonotube fiber forms were utilized for forming round columns of concrete. Information was also available on Sonopipe sleeves and Duropipe.

R. L. Spillman Co., Columbus, O.—Metal forms and machinery for molding concrete specialty products are this company's business, and a few of the several hundred design molds available were on display. According to the company, all molds are made of heavy gage steel and are easy to strip and clean, and allow easy casting.

Water Seals, Inc., Chicago, Ill. — Information and data were available on PRC epoxy compounds and caulking and sealing compounds. The different types of waterstops made by Water Seals were illustrated in a display.

Western Waterproofing Co., Inc., St. Louis, Mo. — Flexible Resto-Crete, a product for weatherproofing buildings, was one of the main points of interest in this booth. Also available was information on Western services, including tuckpointing, building cleaning, concrete restoration, and subsurface water protection.

C. K. Williams & Co., East St. Louis, Ill.—The Williams exhibit was definitely colorful. Featured was a selection of the iron and chromium oxides the company manufactures for mixing with concrete to produce colored concrete products. The oxides are available in a wide range of yellows, buffs, reds, browns, greens, and black.



COLUMBUS STREET SHIP TERMINAL — Charleston, S.C. POZZOLITH concrete deck beams and 10" concrete slab (lower right) are used on top of the pilings for 750,000 sq. ft. dock. Construction Agency: South Carolina State Ports Authority, Charleston • Architect-Engineer: Lockwood-Greene Engineers, Spartansburg, S. C. • General Contractor: J. A. Jones Construction Co., Charlotte • Prestressed Concrete and POZZOLITH Ready-Mix Concrete Supplier: Concrete Materials, Inc., Charlotte.

95 miles of prestressed piles

for new $7.7 million Charleston shipping terminal

Over 500,000 linear feet of 21-inch octagonal prestressed POZZOLITH concrete piling went into the foundation of the main dock of this new Columbus Street Ship Terminal. The 5,000 piles averaged 100 feet in length.

To keep the fast-moving pile-driving operation supplied with up to 30 piles per day—a compact, completely mechanized prestress plant was set up near the job site. Concrete was automatically batched, turbine-mixed and placed into 500 ft. casting beds.

The eighteen 7/16" strands in each pile were tensioned to 18,900 lb. per strand. Piles were steam cured in the beds for 12 hours at 160F to a minimum of 4000 psi, then transferred to a 10-acre yard for 14 days of final curing. Using Type I cement, the zero slump mix was designed for 5000 psi at 28 days, but actual strengths averaged over 7000 psi.

POZZOLITH was used to increase workability, reduce total water content and provide both the durability to salt water exposure and high strength required. Call in the local Master Builders field man to demonstrate how POZZOLITH concrete is superior in quality and economy to plain concrete, or concrete made with any other admixture.

The Master Builders Company
Division of American-Marietta Company
Cleveland 18, Ohio
World-wide manufacturing and service facilities

MASTER BUILDERS. POZZOLITH*

*POZZOLITH is a registered trademark of The Master Builders Co. for its ingredient for concrete which provides maximum water reduction, controls rate of hardening and increases durability.

Local Committee Sponsors Special Events

The nontechnical program was in the hands of the St. Louis Convention Committee. In addition to a full schedule of events for the ladies, the committee sponsored a student architectural competition, an exhibit, and a dinner at which Waldo G. Bowman, editor, *Engineering News-Record*, was guest speaker.

The ladies program included shopping tours, a style show, a trip to the St. Louis Art Museum, and a card party.

Local committee

Local planning for the 57th Convention was under the able direction of General Chairman Howard A. Coleman. Robert A. Willis served as secretary and August A. Lamack served as treasurer.

Other members of the General Committee included Clarence Ax, Albert H. Baum, William C. E. Becker, Sidney Bierman, Neal J. Campbell, George J. Herbst, Raymond F. Powell, and George W. Simms. An Executive Committee coordinating the efforts of the various subgroups included Mr. Coleman, A. Carl Weber, Carl J. Chappell, and Paul K. Nichols.

The various activities were handled by the following people: Exhibits — Paul E. Magoon, chairman; Robert D. Bay, Carl J. Chappell, and William Miller.

Entertainment and Hospitality — George N. Bishop, Jr., chairman; Albert H. Baum, George S. Herbst, Paul K. Nichols, Raymond F. Powell, M. N. Quint, L. L. Schwarz, and A. Carl Weber.

Hotel Arrangements — Bruce Kester, chairman; W. G. Beck, W. C. E. Becker, Sidney Bierman, C. J. Chappell, C. B. Germer, and D. J. Stocker.

Membership—A. H. Brust, chairman; Clarence Ax, W. C. E. Becker, Neal J. Campbell, Henry M. Reitz, and Harold M. Telthorst.

Publicity — Fred B. Schulenberg, chairman; Frank A. Beets, Neal J. Campbell, Paul E. Magoon, and A. Carl Weber.

Ladies Events—J. John Brouk, chairman; Mrs. S. Bierman, Mrs. G. N. Bishop, Jr., Mrs. J. J. Brouk, Mrs. A. H.

Winners of the Student architectural competition pose with Carl J. Chappell of the local committee on the left and retiring President Joe W. Kelly on the right. Award winners are (l. to r.) Richard A. Kimbrough, third prize, Richard R. Bergmann, first prize, and George A. Albers, second prize

Brust, Mrs. N. J. Campbell, Mrs. C. J. Chappell, Mrs. H. A. Coleman, Mrs. G. J. Herbst, Jr., Mrs. B. Kester, Mrs. A. H. Lamack, Mrs. P. E. Magoon, Mrs. P. K. Nichols, Mrs. A. C. Weber, and Mrs. R. A. Willis.

Dinner program

On Wednesday evening, February 22, the local committee sponsored a cocktail hour and dinner. At the dinner, Waldo C. Bowman, editor, *Engineering-News-Record*, presented an illustrated talk "Concrete Construction—World-Wide" in which he presented first-hand observations gathered during his recent around-the-world inspection tour. Among the projects he reported were descriptions of construction activity in Japan, Formosa, and India.

A. Carl Weber, director of research engineering, Laclede Steel Co., was toastmaster for the evening.

LOOKING AHEAD

May 11-13, 1961—Annual Convention, National Lime Association, Grand Hotel, Point Clear, Ala.

May 15-16, 1961 — 41st Annual Meeting, Society of American Military Engineers, Washington, D.C.

May 16-18, 1961—1961 Spring Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

May 22-24, 1961 — Fifth Annual Convention, Construction Specifications Institute, Commodore Hotel, New York

June 12-15, 1961—National Water Resources Research Symposium, Colorado State University, Fort Collins, Colo.

June 25-30, 1961—64th Annual Meeting, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

June 26-July 1, 1961 — 7th International Congress on Large Dams, Rome, Italy

July 3-6, 1961—RILEM Symposium on Durability of Concrete, Prague, Czechoslovakia

July 30-Aug. 13, 1961 — RILEM International Symposium on the Durability of Concrete, Prague, Czechoslovakia

Aug. 30-Sept. 2, 1961 — IASS-RILEM Shell Colloquim, Delft, the Netherlands

Sept. 4-6, 1961 — IASS Shell Colloquim, Brussels, Belgium

Oct. 7-10, 1961—Western Building Industries Exposition, Great Western Exhibit Center, Los Angeles

Oct. 16-20, 1961 — Annual Convention, American Society of Civil Engineers, Hotel Statler, New York

Oct. 15-19, 1961—7th Annual Meeting, Prestressed Concrete Institute, Brown Palace and Cosmopolitan Hotels, Denver, Colo.

Oct. 16-17, 1961—Annual Meeting, The Carolinas Ready Mixed Concrete Association, Inc., Jack Tar Poinsett Hotel, Greenville, S. C.

Nov. 1-3, 1961 — 14th Regional Meeting, American Concrete Institute, Dinkler-Tutwiler Hotel, Birmingham, Ala.

Positions and Projects

Southern California Chapter holds third annual meeting

The third annual meeting of the ACI Southern California Chapter was held on March 16 at the Roger Young Auditorium in Los Angeles.

Chapter president, Dewain R. Butler, president of Integrated, Inc., Los Angeles, presided at the business meeting, presenting the newly elected officers and directors for the ensuing year.

Byron P. Weintz, chief engineer, Consolidated Rock Products Co., Los Angeles, is chapter president for '61 with Henry M. Layne, consulting and structural engineer, Los Angeles, as vice-president.

Newly elected directors for 3-year terms are: E. A. Peterson, vice-president and manager, Rocklite Products Inc., Ventura; and Edward K. Rice, partner, T. Y. Lin & Associates, Van Nuys, Calif. C. Dubbs and E. M. Markell completed terms as directors as of the March meeting.

SAMUEL HOBBS, secretary-treasurer of the ACI Southern California Chapter, presenting the ACI Wason Medal for Most Meritorious Paper to ROBERT A. WILLIAMSON, chief structural engineer, Holmes and Narver, Inc., Los Angeles, for his paper, "Performance and Design of Special Purpose Blast Resistant Structures," published in the May, 1960, ACI JOURNAL.

Presentation of the ACI Wason Medal for Most Meritorious Paper of 1960 was made to Robert A. Williamson, chief structural engineer, Holmes and Narver, Inc., Los Angeles, for his paper, "Performance and Design of Special Purpose Blast Resistant Structures," which was published in the May, 1960, ACI JOURNAL. Mr. Williamson was not able to be present at the Awards Luncheon on February 22, in St. Louis, when Joe W. Kelly, retiring president, presented the ACI annual awards in behalf of the Board of Direction of the Institute.

Program Chairman Robert E. Tobin, Portland Cement Association, Los Angeles, presented a technical program on a discussion of equipment, methods, developments, techniques and good practice for quality construction with Shotcrete or Gunite. Featured speakers were John C. Fredericks, manager, Gunite Contractors Association, and C. Taylor Test, applications research engineer, Riverside Cement Co., Los Angeles.

Samuel Hobbs, secretary-treasurer of the chapter, announced five new chapter members: Robert Lewis Collins, structural engineer, Los Angeles Board of Education; Delbert L. Jex, concrete contractor; Hector P. Mecca, inspector, Los Angeles Housing Authority; Robert Pfaff, Burns and Roe, consulting engineers, Santa Monica; and Joseph P. Oeland, project superintendent, George A. Fuller Co., Los Angeles.

PCA assigns Claybrook to Nashville office

Harry D. Claybrook has joined the Portland Cement Association as general field engineer with headquarters in Nashville. He has been associated with the Nashville architectural firm of Marr and Holman, as a structural engineer for the past 11 years.

Northern California Chapter installs '61 officers

Installation of new chapter officers took place at the second annual meeting of the ACI Northern California Chapter on March 8 in Berkeley.

Milos Polivka, associate professor of civil engineering, University of California, Berkeley, is the new chapter president succeeding Harry E. Thomas, Pacific Gas and Electric Co., Emmeryville. M. R. Neumann, vice-president, Testing Engineers, Inc., Oakland, is the new vice-president succeeding Professor Polivka. Wilbur E. Moulton, testing engineer, Pacific Cement and Aggregates, Inc., San Francisco, continues as secretary-treasurer of the chapter.

Newly elected directors include Orville Jack, production control manager, Permanente Cement Co., Oakland, and Ivor Thomas, office engineer, K. J. Kaiser Co., Oakland. Continuing directors are: J. A. Dobrowolski, E. L. Howard, R. T. Retz, and D. K. Woodin. Retiring directors were J. E. Jellick and Mr. Neumann.

Ellis L. Armstrong, president, Better Highways Information Foundation, Washington, D.C., was principal speaker for the meeting, speaking on portland cement concrete and the national highway program.

Milos Polivka, left, succeeds Harry E. Thomas, right, as president of the Northern California Chapter

Mr. Armstrong's wide experience includes 18 years with the U. S. Bureau of Reclamation on design and construction of dams, railroads, highways, and related projects. In 1953 he served as engineer member of the Egyptian-American Rural Improvement Commission, Cairo, Egypt. He also acted as special consultant to the Egyptian government on High Aswan Dam project.

Technical committee chairman appointments

Acceptance has been received by the following new committee chairmen appointed by the Board of Direction at the 1961 annual meeting in St. Louis, February 20-23.

Committee 215, Fatigue of Concrete
Thomas E. Stelson, professor and head, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, succeeds Gene M. Nordby, head, Civil Engineering Department, University of Arizona, Tucson.

Committee 328, Limit Design—Joint ACI-ASCE
Herbert A. Sawyer, professor, Department of Civil Engineering, University of Connecticut, Storrs, succeeds Alfred L. Parme, structural engineer, Portland Cement Association, Chicago.

Committee 404, Joint Sealants
George B. Wallace, civil engineer, U. S. Bureau of Reclamation, Denver.

Committee 622, Formwork for Concrete
Joseph R. Proctor, Jr., estimating engineer, Dravo Corp., Contracting Division, Neville Island, Pittsburgh, succeeds Harry Ellsberg, consultant, Giffels and Rossetti, Detroit.

Committee 716, High Pressure Seam Curing
George W. Washa, chairman, Department of Mechanics, University of Wisconsin, Madison, succeeds Samuel B. Helms, research engineer, Lehigh Portland Cement Co., Coplay, Pa.

Reappointments

Acceptance has been received by the following committee chairmen who have been reappointed for 1 year because of work in progress.

Committee 322, Design of Structural Plain Concrete
Paul Rogers, Paul Rogers and Associates, Inc., Chicago

Committee 335, Deflection of Concrete Building Structures
M. V. Pregnoff, Pregnoff and Matheu, San Francisco

Committee 621, Aggregates
William R. Waugh, chief, Concrete Branch, Engineering Division, Chief of Engineers, Department of the Army, Washington, D.C.

Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE
Jack R. Janney, The Engineers Collaborative, Chicago

Hawaii joins ACI chapter ranks

The charter for the ACI Hawaii Chapter will be presented at their meeting on June 7. The presentation will be made by ACI president Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento.

The petition of 30 Institute members in Hawaii to form the chapter was approved by the Board of Direction on February 19.

Board authorizes formation of committee on joint sealants

The Board of Direction has authorized the formation of a new technical committee designated as ACI Committee 404, Joint Sealants.

George Wallace, civil engineer, U. S. Bureau of Reclamation, Denver, Colo., was appointed chairman of this new committee.

The committee will study, assemble, and report information on materials and methods used for sealing joints and cracks in concrete structures. Recommendations will be developed for use of sealants, caulking compounds, waterstops, and other materials used to seal joints. The committee will also promote research to determine suitable materials, details of joint design, and methods of sealant installation for effective performance.

Concrete conferences in Utah and Minnesota

ACI was represented at the recent Fifth Head-of-the-Lakes Concrete Conference at the University of Minnesota, Duluth, and the Third Annual Concrete Conference at Utah State University, Logan, by Technical Director Kenneth D. Cummins.

University of Minnesota

On March 9 the Fifth Head-of-the-Lakes Concrete Conference was held in the Kirby Student Center Building on the University of Minnesota campus in Duluth. The conference was sponsored by the Portland Cement Association in cooperation with the University.

PCA engineers presented the following papers: "Design and Construction of Load-Bearing Concrete Masonry Walls," by Ralph E. Spears, regional structural engineer, Kansas City; "Lightweight Structural Concrete," by J. J. Shideler, manager, Products and Applications Development Section, Skokie; "Observations on the AASHO Test Road," by O. O. Pfutzenreuter, field engineer in Duluth; "Residential Street Paving and Sidewalk Construction," by Byron G. Sneva, field engineer in Minneapolis; "Forming and Construction Procedures on Minnesota Shell Roof Projects," by R. W. Randall, district structural engineer in Minneapolis.

The two PCA films "Casting and Erecting Concrete Curtain Walls" and "How to Build New Shapes in Concrete" were also presented. Norman E. Henning, vice-president of engineering, Twin City Testing and Engineering Laboratory, presented a talk on "Design and Control for Quality Concrete." Mr. Cummins presented a report on "Recommendations for General Formwork" based on the report of ACI Committee 622, Formwork for Con-

crete, which appeared in the March, 1961, ACI JOURNAL. About 200 attended the conference.

Utah State University

On March 14 the Third Annual Concrete Conference was conducted at Utah State University, Logan.

H. L. Day, materials engineer for the Idaho Highway Department, Boise, presented a talk on prestressed concrete in highway construction. Chris Pickett, structural engineer for Portland Cement Association, Salt Lake City, discussed curing methods and materials. Lightweight aggregate concrete in modern construction was covered by Ivan L. Lynn, head, Concrete Research for Ideal Cement Co., Ft. Collins, Colo., and Arthur G. Wood, sales engineer, Idealite Co., Denver.

Vaud E. Larson, assistant project construction engineer, U. S. Bureau of Reclamation, Denver, gave an illustrated talk on the concrete construction at Glen Canyon Dam. Elliot Rich, professor, Utah State University, gave a brief synopsis of the analysis of stresses and load carrying capacity of Lost River Bridge in Idaho. Mr. Cummins discussed forms and formwork for concrete structures.

About 250 attended the conference including students, contractors, engineers and suppliers from Utah and southern Idaho.

German Concrete Conference for '61 held in Berlin

The German Concrete Conference for 1961 was held in Berlin, March 22-24.

The technical program featured research experts and engineers speaking on various phases of concrete technology. Three of the speakers devoted their papers to the discussion of German building accomplishments abroad. Other speakers discussed technical viewpoints in the construction of city highways, reinforced precast units, and properties of concrete. About 15 papers were presented during the 3-day meeting.

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Michigan Chapter discusses truck mixing at March meeting

Attendants at the March 8 meeting of the ACI Michigan Chapter heard a talk by Emil Baugh, sales manager, Truck Mixer Division, Jaeger Machine Co., Columbus, Ohio, "The Art of Mixing Concrete in a Truck Mixer Drum." Mr. Baugh traced the mixing of concrete from hand operation to present-day mixing trucks. He described the end-to-end, figure eight motion of hand mixing by hoe or shovel and showed how this action was incorporated into truck mixer design.

The earliest truck drums with blades were used in conjunction with central mix plants. The truck was used for hauling premixed concrete and the rotating blades were used to aid discharge. Mr. Baugh traced the design of blades in truck mixers from the protruding "hoe" type to the spiral blades in combination with "throw back" and discharge blades. The importance of blade angle, height, and pitch were noted. He also covered the design of drums and their capacity. He noted the effect on mixing of the length and angle of the discharge cone.

Missouri concrete conference

The Department of Civil Engineering, University of Missouri School of Mines and Metallurgy in cooperation with the Missouri Ready-Mixed Concrete Association and the Portland Cement Association conducted the Missouri Concrete Conference, February 16-17, at Rolla, Mo. E. W. Carlson, Civil Engineering Department chairman at the University, acted as conference director.

William A. Maples, Institute secretary-treasurer, was guest speaker at a luncheon on February 17, speaking on "Current Activities of the American Concrete Institute."

The 2-day program spanned a variety of technical subjects covering highway and street paving problems, concrete usage on the farm, color and concrete, and reinforcing steel. Other technical papers presented dealt with ready-mixed concrete, hot and cold weather concreting, outstanding concrete structures in Europe, and a report on the construction of a thin-shell shopping center.

In addition to Mr. Maples, four ACI members participated in the program: George R. Bathe, president, Ready-Mixed Concrete Co., Omaha; Robert W. MacDermott, structural engineer, St. Louis; H. C. Rose, general field engineer, Portland Cement Association, St. Louis; and A. Carl Weber, director of research engineering, Laclede Steel Co., St. Louis.

Rowe joins Vanderbilt faculty

Robert S. Rowe, formerly head of the Civil Engineering Department at Duke University, Durham, N. C., is now Dean of the Engineering Department at Vanderbilt University, Nashville, Tenn.

Fire resistance rating for prestressed double-tee members

A 2-hr fire resistance label will now be provided by Underwriters' Laboratories for prestressed concrete double-tee floor and roof members, based on the results of fire tests sponsored by the Prestressed Concrete Institute.

Minimum bottom cover specified for UL label approval is 1⅝ in. and minimum side cover is 1⅛ in., according to Norman L. Scott, PCI executive secretary. He explained that these specifications cover virtually all prestressed double-tee and channel sections now available in the United States and Canada.

Up to five strands per stem may be used in UL-labeled members. All commonly used strand sizes are permitted, ¼ to ½ in. Maximum width of members is established at 60 in., and the depth and length are unrestricted.

new books for the active engineer:

Design of Prestressed Concrete Beams—#365

by William H. Connolly. Presents a simplified, rational, and clear-cut method for the design of pretensioned and post-tensioned concrete members. Using its design tables, you can reduce the tedious trial and error normally involved in the selecting of the cross-section. Extremely practical, it places emphasis on elastic design for bending, and analyses prestress losses, shear, differential shrinkage, and ultimate flexural strength. Based on report of ACI-ASCE Joint Committee 323 "Tentative Recommendations for Prestressed Concrete."—264 pages, 6" x 9", $11.50

- Explicit, practical instructions
- Over 120 illustrations of stress diagrams and cross-sections
- Tables for selecting flange widths for I-beams and T-beams
- Tabulated solution for I-beams beyond the critical span
- Table of design constants for composite sections

Elementary Statics of Shells—#692

- Visual presentation of the spatial interplay of forces in shells
- Approximate method for the calculation of shells of revolution subjected to loads of rotational symmetry
- 19-page appendix with tabulated values of the solutions of the Membrane Theory
- Over 125 clear line illustrations

by Alf Pflüger. Newly translated from the German, this compact work offers a simple and methodical introduction to shell theory and design. Specially adapted to the needs of the practicing structural engineer, its approach is more elementary than recent studies that employ highly advanced mathematical tools. Theory is elaborated only when necessary and all given equations are capable of visual interpretation. Once the basic equations are set down, their results can be practically interpreted in the tables and graphs.—122 pages, 6" x 9", $8.75

Reinforced Concrete Column Tables, Ultimate Strength Design—#495

by Hugh F. Fenlon. With this book, the column designer can confidently select reinforced concrete columns for every set of conditions. Following the ACI building code, the 300 tables cover columns up to 24" square and 36" in diameter. In all, there are 8,568 time- and money-saving designs in four material strength combinations. Column loads, maximum allowable loads, moment capacities, and the size and arrangement of reinforcing bars are given. 316 pages, 8½" x 11", $15.00

also of interest—

#655—Structures, by Pier Luigi Nervi. 118 pages, 7¼" x 9⅞", Illus. $6.95

#760—The Structures of Eduardo Torroja. 198 pages, 7" x 9¾", Illus., $8.50

#595—Applied Structural Design of Buildings, by Thomas H. McKaig, (2nd Edition). 442 pages, 7⅛" x 10", Illus., $12.50

#610—Field Inspection of Building Construction, by Thomas H. McKaig. 337 pages, 6" x 9", Illus., $9.35

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Construction techniques spotlighted at Iowa concrete conference

Construction practices and techniques received major emphasis at the Twelfth Annual Better Concrete Conference, March 8 and 9, 1961, at Iowa State University, Ames. Despite a late winter blizzard of major proportions more than 100 engineers, technicians, and producers from all over Iowa attended the 2-day conference presented by the Department of Theoretical and Applied Mechanics and Engineering Extension of Iowa State University.

Sessions were devoted to the use of epoxies and latex in concrete construction and repair, formwork for concrete, precast and prestressed concrete, and durability.

The development and use of epoxies and latex materials in concrete construction and repair was discussed by R. Douglas Eash, coatings field development chemist, Dow Chemical Co., Midland, Mich. Experience with epoxies and latex in highway construction was discussed by Clarence DeYoung, materials engineer, Iowa Highway Commission, Ames. The repair of heavy duty concrete floors was described by Charles E. Wittmack, Jr., manager, ready-mixed concrete division, Arthur H. Neumann and Bros., Inc., Des Moines, Iowa.

Another session was devoted to formwork. Ted Hunt, decorative concrete specialist, Portland Cement Association, Chicago, described plastic forms and illustrated precast products as well as cast-in-place construction where plastic forms had been used. Forming materials and techniques were described by Thomas J. Reading, materials engineer, U. S. Corps of Engineers, Missouri River Division, Omaha, Neb. H. A. Welton, Younglove Construction Co., Sioux City, Iowa, illustrated various details of slip-form construction and also demonstrated various forming techniques and equipment. ACI Technical Director Kenneth D. Cummins discussed recommended practices for formwork.

Current precast and prestressed concrete construction in Iowa was described and illustrated by H. J. Jobse, structural engineer, Portland Cement Association, Des Moines.

The subject of durability received considerable attention. The problem and its solution were discussed in detail by Herbert K. Cook, vice-president for engineering, Master Builders Co., Cleveland. Good techniques in placing and finishing flat slab work were emphasized by J. C. Yeager, construction superintendent, Portland Cement Association, New York. Responsibility of the ready-mixed concrete operator in achieving durable concrete was the topic of Kenneth H. Johnson, manager, J. C. White Concrete Co., Des Moines, and durable concrete highways were discussed by Thomas E. McElherne, assistant materials engineer, Iowa Highway Commission, Ames.

Also on a practical level was a demonstration by Elbert R. Richardson, field engineer, Master Builders Co., Bellevue, Neb., on the use of air meters and yield buckets.

Current practice in concrete construction and some of the projects now under construction were described by John H. Banker, construction superintendent, Portland Cement Association, Chicago.

Luncheon speakers for the 2 days were Prof. Sam Legvold, professor of physics, Iowa State University, who discussed frigid physics and Robert E. Wilde, ACI assistant secretary, who described recent Institute activities and illustrated some of the concrete projects in the Detroit area.

Other co-sponsors of the conference along with Iowa State University were ACI, Portland Cement Association, Iowa Highway Commission, Iowa Ready-Mixed Concrete Association, American Institute of Architects (Iowa), Master Builders of Iowa, and Associated General Contractors of Iowa.

MIT to present 2-week summer experiment techniques program

Massachusetts Institute of Technology will present a 2-week special summer program on experimental techniques, designed particularly for industrial people in the fields of research and development, June 13-23 inclusive. The course will consist of lectures and laboratories arranged to provide the necessary background to make and interpret measurements in the general mechanical engineering field.

Measurements to be discussed include: displacement, strain, force, torque, velocity, acceleration, flow, temperature, wear, and surface characteristics. Methods for making measurements include: optical, mechanical pneumatic, electric (resistive, inductive, capacitive), photographic, and radioisotope techniques. The dynamical aspects of measurement and statistical interpretation of results will be covered.

Williams appointed NSA technical committee head

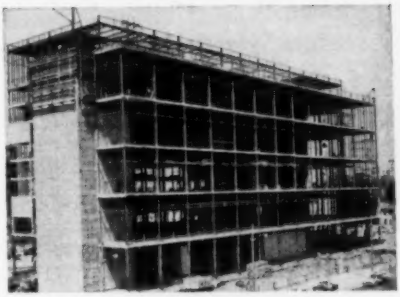
H. T. Williams, consulting engineer, Standard Slag Co., Youngstown, Ohio, has been named chairman of the National Slag Association's technical committee.

Dundee service center receives landscape award

The Dundee Cement Co.'s distribution and service center in Chicago, has been recognized by the South Chicago Chamber of Commerce with an award in the area's rehabilitation and beautification program.

The Dundee facility received a citation and U.S. Savings Bond for major landscape improvements and the confidence displayed in the continuing growth of South Chicago.

The presentation was made to John Bader, Chicago district sales manager, by M. P. Staniec, vice-president of the SCCC.



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J-51

National Water Resources Research Symposium to be held in Colorado, June 12-15

The critical needs for basic engineering research in water resources development are to be explored at a national symposium to be held June 12-15, on the campus of Colorado State University at Fort Collins, Colo.

The symposium is being sponsored by the Bureau of Reclamation of the Department of the Interior, the American Society of Civil Engineers and Colorado State University.

Engineering and scientific specialists will participate in seven panel discussions on needed research in engineering for water resource projects. The panelists, comprising from five to ten members for each panel, will represent educational institutions, Government agencies, selected State agencies, private concerns, and other organizations in the United States and from foreign countries.

One of the panel discussions will be devoted to "Physics and Chemistry of Static and Dynamic Behavior of Concrete," with Douglas McHenry, director, Research and Development Division, Portland Cement Association, Skokie, Ill., as panel chairmen. Panel members are: Roy W. Carlson, consulting engineer, Berkeley, Calif.; Bryant Mather, Waterways Experiment Station, Corps of Engineers, Jackson, Miss.; Ralph L. Bloor, International Bank of Reconstruction and Development, Washington, D.C.; Clyde E. Kesler, College of Engineering, University of Illinois, Urbana, Ill., Lewis H. Tuthill, Division of Design and Construction, California State Department of Water Resources, Sacramento, Calif.; T. C. Powers, Portland Cement Association, Skokie, Ill.; Phil M. Ferguson, Civil Engineering Department, University of Texas, Austin, Tex., Raymond E. Davis, Engineering Materials Laboratory University of California, Berkeley, Calif., L. G. Puls, Bureau of Reclamation, Denver, Colo., and Walter H. Price, Bureau of Reclamation, Denver, Colo.

1962

The Technical Activities Committee has begun its plans for ACI's 58th annual meeting, March 12-15, Denver.

Those who are interested in participating in the program should write to Institute headquarters before July 1, furnishing a synopsis giving the scope of the proposed paper and indicating features that the author thinks will justify its inclusion in the technical program.

Preliminary drafts should reach TAC for approval and acceptance by September 15; final manuscripts will be due by Jan. 1, 1962.

Lobaugh retires from Universal Atlas Cement

Frank E. Lobaugh, manager of Lumnite Sales and Service, Universal Atlas Cement Division of United States Steel, New York, has retired after 20 years of service.

He joined Universal Atlas in 1941 as a member of Lumnite Technical Development and Sales. He was made technical service director in 1944 and was appointed manager of Lumnite Sales and Service in 1947.

Burns opens Houston engineering office

Bennett W. Burns has announced the opening of offices in Houston, Tex., for the practicing of civil, structural, and mechanical engineering.

Mr. Burns, formerly managing partner of the Houston office of the firm of H. E. Bovay, Jr., consulting engineers, had been with the Bovay firm for the past 9 years. Prior to that he was with the Humble Oil and Refining Co. for 16 years in various design capacities. He is a graduate of Rice Institute, Houston.

Woodward appointed regional field sales manager

Mark R. Woodward has been appointed to the post of regional field sales manager of the east central division of The Master Builders Co., Cleveland.

Mr. Woodward joined Master Builders in 1949 and was appointed manager of the firm's Buffalo office in 1951. In his new post he will be responsible for the management and direction of the Master Builders sales staff in the east central states.

Huron names new director of research and development

Richard E. Galer has been named to succeed the late Francis A. McAdam, as director of research and development for Huron Portland Cement Co., Detroit.

Formerly chemical engineer of Huron's Alpena manufacturing plant, Mr. Galer is a 1949 graduate in chemical engineering of Michigan State University, East Lansing. Prior to joining Huron in January, 1960, Mr. Galer had been a member of the production research staff at Wyandotte Chemicals Corp. for 4 years. His experience prior to that included association with Detrex Chemical Co., Detroit; Lapaco Chemicals Co., Lansing; and Leonard Refineries in Alma, Mich.

Kleine-Kracht affiliates with Louisville firm

Joseph A. Kleine-Kracht has announced his affiliation with the Louisville consulting engineering firm of Hummel and George. The new firm is known as Hummel, George and Kleine-Kracht, Inc., consulting engineers, Louisville.

Dundee Cement starts second terminal in Chicago area

Dundee Cement Co. is constructing a new $600,000 distribution and service center in the Northwest Chicago area. The new terminal, the second such installation for this area, is being built on a 7½ acre site in Des Plaines, Ill. Construction of the new facility will be completed early this spring.

Student competition

(Continued from p. 20)

The project was a memorial chapel for an island naval base having a capacity of 400 persons. There were three awards given: $250, $100, and $50 for first, second, and third places, respectively.

There were 15 entries for the competition all by first-semester fifth architectural students. The three winners were, in order, Richard R. Bergmann, University of Illinois, for his design for an interdenominational chapel; George A. Albers, University of Illinois, for his design of a Protestant chapel; and Richard A. Kimbrough, University of Illinois, for his design of a Catholic chapel.

The judges were George Anselevicius, associate professor of architecture, Washington University, St. Louis; Jack S. Baker, professor of architecture, University of Illinois, Urbana; Rex L. Becker, AIA, Froese, Maack, and Becker, St. Louis; Arnold H. Bock, structural engineer, Portland Cement Association, St. Louis; and Frederick Dunn, AIA, Frederick Dunn and Associates, Architects, St. Louis.

HRB preparing report on AASHO road test

The Highway Research Board of the National Academy of Sciences — National Research Council, Washington, D.C., announces that the reports of research findings of the AASHO Road Test will be published on the completion of the data analyses, which will require about 6 months more.

Two preliminary reports are being prepared and should be available in about 3 months. The first will cover the background and concepts of the project; the second will describe in detail the construction of the test facilities. Both reports will form a basis for complete understanding of the final reports, which will cover the technical data produced during the test.

The Road Test, sponsored by the American Association of State Highway Officials, is a scientific study of the performance of highway pavements and bridges of various designs when subjected to repeated passages of vehicles applying known loads.

Schnarr retires from Ontario Hydro

After more than 43 years' service with Ontario Hydro, Wilfrid "Bill" Schnarr has retired. He had been engineer-in-charge of the Concrete Control Department since 1949, when the Research and Testing Department became the Research Division. For practically his entire career he has been closely associated with all of Ontario Hydro's construction work involving concrete, having had primary responsibility for the quality control of more than 8 million cu yd of concrete.

Through his long years of experience he has made important contributions to numerous advances in the use of concrete. Five of his papers have been published in the ACI JOURNAL.

A member of ACI since 1925, he has served on a number of Institute technical committees, including those responsible for the Standard on Recommended Practice for Winter Concreting (ACI 604-56) and for early editions of the *ACI Manual of Concrete Inspection*.

Burmeister appointed to new post

R. A. Burmeister, formerly materials engineer, City of Milwaukee Testing Laboratory, has been named engineer-in-charge, Construction Division, City of Milwaukee Department of Public Works, Bureau of Engineers.

Prestressed floor system passes 2 hr fire test

West Allis Concrete Products Co., Spancrete Division, sponsored its third successful fire test on February 16 at Underwriters' Laboratories, Northbrook, Ill.

The floor system tested consisted of a 6 in. machine manufactured prestressed concrete cored slab topped with 2 in. of cast-in-place concrete. The slabs and topping were made of sand-gravel concrete.

A 146 lb per sq ft superimposed load was applied to the slab. The assembly was subjected to the ASTM standard time-temperature curve for 2 hr and 5 min without structural failure. The strand which had a concrete cover of ¾ in. reached a maximum temperature of 1250 F and the maximum deflection was 0.46 in. after 2 hr. Although not required by ASTM standards the slabs were also subjected to the hose stream test and the double live load test. The assembly easily passed both of these optional tests.

Results of this test will add another report to the rapidly growing library on the fire resistance of prestressed concrete. Spancrete is one of the early members of the Prestressed Concrete Institute which is conducting an extensive fire testing program. The Portland Cement Association, Skokie, Ill., is also gathering fire test information on prestressed concrete at its fire research laboratory.

New book tells

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Lincer joins New York consulting firm

Maxwell Lincer has been admitted to general partnership in the consulting engineering firm of Weinberger, Frieman, Leichtman and Quinn, New York.

Corbetta addresses New York CIB members

ACI Vice-President Roger H. Corbetta, president of the Corbetta Construction Corp., New York and Chicago, was guest speaker at the March meeting of the Concrete Industry Board of New York City. Mr. Corbetta's subject was "Are We Prepared for the Future of Concrete."

Mr. Corbetta was the first president of CIB and served in that capacity for 5 years. Since then he has been a member of the Board of Directors as ACI representative.

WPI schedules series of 2-day fallout shelter design workshops

Worcester Polytechnic Institute, Worcester, Mass., will conduct a series of at least three 2-day fallout shelter design and evaluation workshops for architects and engineers before the end of June. The workshop locations have yet to be designated, according to Arthur B. Bronwell, WPI president, but they will be in the Office of Civil and Defense Mobilization Region One, which includes the New England states, New York, New Jersey, Puerto Rico and the Virgin Islands.

The workshops will be a part of a nationwide series to be conducted by eight universities or colleges, one in each OCDM Region. All will teach the fundamentals in design of shelter to protect against gamma radiation which would result from nuclear attack.

Bates speaks on construction industry methods in Russia at two April joint meetings

A. Allan Bates, vice-president, Portland Cement Association, Chicago, was guest speaker at a joint meeting of the American Society for Testing Materials, New York District; American Society of Civil Engineers, Metropolitan Section; and the Concrete Industry Board Inc. of New York, April 19, in New York.

Dr. Bates, who recently visited Russia as part of the information exchange agreement between America and Russia, spoke on observations on construction methods being used in the Soviet Union.

On April 25 he addressed the joint meeting of the American Society for Testing Materials, Detroit District; American Society of Civil Engineers, Michigan Section, and the American Concrete Institute, Michigan Chapter, in Detroit. He discussed the role of construction materials and methods in the American and Soviet future. His talk was based on his long experience in many materials technologies and on observations made on his recent trip in the Soviet Union.

Currently, Dr. Bates is serving on the ACI Board of Direction. He is also currently national president of ASTM and is a member of the executive committee of the Building Research Advisory Board of the National Academy of Sciences.

BRI announces topics for spring meeting

Major technical programs on adhesives and sealants, plastics, public entrance doors and new building research will be featured at the 1961 Spring Conferences of the Building Research Institute, May 16-18, Shoreham Hotel, Washington, D.C.

The 3-day meeting is the fourth in the newly inaugurated series of BRI semiannual multisubject conferences. One of the sessions will be devoted to Requirements for Weatherproofing Thin Shell Concrete Roofs and will include reports on weatherproofing and sealant materials, the effect of physical factors, and properties and uses of available materials. At another session, Selection and Field Application of Adhesives, several important papers will be presented on adhesives for ceramic, plastic and metal tile, wallboard, resilient floor coverings, insulation, plastic laminates, acoustical tile, and new field applications. Architects and engineers will find these sessions of value in bringing themselves up-to-date with new technological developments. The meeting is open to the interested public as well as to BRI members and guests.

Yamasaki among 1961 AIA award winners

Oft-honored Minoru Yamasaki, principal of the firm of Yamasaki and Associates, Birmingham, Mich., was named a 1961 Honor Award winner by the American Institute of Architects for his design of the Reynolds Metals Regional Sales Office Building in Detroit. Ammann and Whitney, New York and Milwaukee, served as structural engineers for the project.

A jury of outstanding architects selected seven recently completed buildings from 270 submissions. Other award winning designs were the Fernando Rivera Elementary School, Daly City, Calif. and the Nuclear Reactor in Rehovot Israel. ACI member Isadore Thompson was the structural engineer on the California school project; ACI member Lev Zetlin was the structural engineer on the reactor project in Israel. Four other structures cited for first honor awards were: the Shrine in New Harmony, Ind.; United States Embassy in New Delhi, India; Summer House in Northville, Mich.; and the Pepsi-Cola World Headquarters in N.Y.

Six ACI members were connected with "Award of Merit" structures: Albert E. Erkel & Associates, structural

engineers on the Ivory Tower Restaurant, Santa Monica, Calif.; I. M. Pei & Associates, architects for the Denver Hilton Hotel, Denver; Chin and Hensolt, structural engineers for the Marin Bay Display Pavilion, San Rafael, Calif.; Isadore Thompson, structural engineer for the Crown Zellerbach Building, San Francisco, Calif.; and Henry A. Pfisterer, structural engineer for the Lincoln Commons Building, Lake Erie College, Painesville, Ohio.

Presentations to the architects and owners of the awarded structures were made at an Awards luncheon, April 26, during the AIA annual convention in Philadelphia.

Dewey and Almy appoints Dodson to research staff

Vance H. Dodson, Jr. has joined the Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge, Mass., as research manager for construction chemicals and rock products chemicals.

Dr. Dodson was formerly research manager of the Battery Division, Electric Autolite Co., Toledo. At Dewey and Almy he succeeds A. Byron Adams, who was recently appointed technical director of the Construction Chemicals Sales Department.

Swanson appointed to New Universal Atlas post

Harry T. Swanson, formerly manager of Atlas White Sales, Universal Atlas Cement Division of United States Steel, New York, has been appointed manager of the newly formed department, Atlas White and Lumnite Sales. Universal has merged the Lumnite Sales and Service Department with the Division's Atlas White Cement Sales. Mr. Swanson joined Universal Atlas in 1948 shortly after his graduation from the University of Minnesota.

Bruno Chiari, formerly Lumnite field engineer, New York headquarters, has been named assistant manager of the department.

J. J. Gould

John J. Gould, partner in the San Francisco consulting engineering firm, Gould and Degenkolb, died of a heart attack on January 30.

A native of Switzerland, Mr. Gould was a graduate of the Engineering School of Zurich. He came to the United States in 1923, and after two years in New York working for consulting engineers and contractors, he transferred to California. There he was the design engineer in charge of the office of L. H. Nishkian, consulting engineer, and later a structural engineer for the state.

In 1935 Mr. Gould became chief structural engineer for the San Francisco Bay Exposition. He was in charge of the structural features of both the temporary Fair and permanent construction on Treasure Island.

Mr. Gould opened a consulting engineering office in 1940. By 1956 his firm had designed over $250 million worth of construction, mostly buildings. H. J. Degenkolb became his partner in 1956.

William H. Armstrong

William H. Armstrong of the consulting engineering firm of Armstrong and Edwards, Atlanta, died recently.

Mr. Armstrong had been a member of the Institute since 1949. He set a record for member-getting as sponsor of 59 new ACI members in 1955. He served as co-chairman of the 1955 regional meeting in Atlanta and was active in technical committee work.

Beginning his career in 1925, while a student of civil engineering at Ohio Northern University, Ada, he was employed by the New Jersey State Highway Department. Later he worked for W. C. Spiker, Atlanta. Since 1931, he had engaged in private practice, working principally on building frames and foundations with some work on bridges and other structures throughout the United States and abroad.

Honor Roll

Jan. 1 — Mar. 31, 1961

Have you signed a new ACI member this month? Among your acquaintances working in concrete, there are probably several excellent ACI prospects who would be only too anxious to join if they knew more about ACI services. Why not tell them about ACI today and have your name listed in next month's honor roll.

Point System

1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

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W. E. Moulton	6
K. K. Nambiar	6
J. Spinel L.	6
E. S. Vieser	6
K. D. Hansen	5
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P. W. Abeles	4½
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W. S. Cottingham	4
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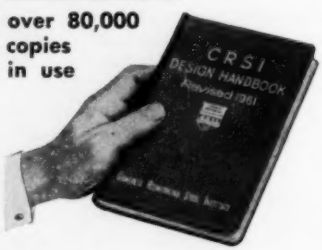

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New Members

The Board of Direction approved applications in the following categories: 78 Individual, 2 Corporation, 19 Junior, and 34 Student, making a total of 133 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,388.

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Berggren, Nelson B., Dillon, Colo. (Off. Fld. Engr., Tipton & Kalmbach)

Black, Wilbur C., Kansas City, Mo. (Desr., Howard, Needles, Tammen & Bergendoff)

Blain, D. L., Knoxville, Tenn. (Matls. Engr., Lambert Bros. Div.)

Bugan, Anton, Roma, Italy (Cons. Engr.)

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Butler, Harold F., Jr., Charleston, W. Va. (Brdg. Insp., W. Va. State Rd. Comm.)

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